SBCN# :2013-13
DATE: 04/16/13
Requested by: M. Katkaphan

SYSTEMWIDE BASELINE CHANGE NOTICE (SBCN)

DOCUMENT/TITLE/NUMBER/REVISION:

Metro Rail Design Criteria Section 5 Appendix Metro Supplemental Seismic Design Criteria Revision 5

CHANGE IMPACT ASSESSMENT SUMMARY: (Attach written explanation of impacts identified)

SCHEDULE ISSUES?:	N	OTHER DOCUMENT REVISIONS REQUIRED?:			
ROM (RANGE):	**NO COST**	DESIGN ISSUES?:	N	COST RECOVERY POTENTIAL:	N
TIME IMPACT:	N/A	SAFETY ISSUES?:	N	OTHER CONTRACTS/PROJECTS?:	N/A
CAL DAYS	N/A	THIRD PARTY?:	N		

Related Request(s)-For-Change: **NONE**

JUSTIFICATION (including benefit or impact if not pursued):

The Revision 5 Metro Rail Design Criteria Section 5 Appendix Metro Supplemental Seismic Design Criteria is to clarify the seismic demands, modeling analyses, and design approach for underground structures.

Chapter 3 Part B7.0 Bore Circular Tunnels is revised; Part B8.1.2 Numerical Modeling Approach is revised; Part B8.1.3 Numerical Deformation Analysis Method is added; Figures 3B-7 replaced; Figure 3B-8 added; Figure 3B-12a added; and Figure 3B-12b revised

PROJECTS/CONTRACTS AFFECTED: For new projects only

PROJ CONTRACT CN # ACTION STATUS

TOTAL ESTIMATED CHANGE COST: (DIRECT)
TOTAL ESTIMATED CHANGE COST: (INDIRECT: POTENTIAL COST RECOVERY)
TOTAL ESTIMATED CHANGE COST: (INDIRECT+ DIRECT)

RECOMMENDATION AND APPROVAL SIGNATURES: (R = RECOMMEND, A = APPROVE)

RTG	APPROVAL	NAME/TITLE	SIGNATURE	DATE
R	DIRECTOR CONFIGURATION MANAGEMENT	D. CURZON		
R	DIRECTOR PROJ. ENG. FACILITIES	A. DAVIDIAN		
R	DIRECTOR PROJ. ENG. SYSTEMS	M. RATNASINGHAM		
R	INTERIM EXECUTIVE DIRECTIOR, RAIL OPERATIONS	B. SHELBURNE		
R	DIRECTOR QUALITY MANAGEMENT	W. MOORE		
R	EO, PROJECT ENGINEERING *	S. MAYMAN	[signature]	05/20/2013
A	EXECUTIVE DIRECTOR TRANSIT PROJECT DELIVERY	K.N. MURTHY		
A	IMPACTED PROJECT MANAGER – N/A			

* To be used by RC & Westside projects
(Circulation to other stakeholders to proceed)

[signature]

METRO RAIL DESIGN CRITERIA

SECTION 5

STRUCTURAL/GEOTECHNICAL

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STRUCTURAL/GEOTECHNICAL

5.0 GENERAL

5.1 INTRODUCTION

5.1.1 Scope

The criteria and codes specified herein shall govern all matters pertaining to the design of Los Angeles County Metropolitan Transportation Authority (Metro) owned facilities including bridges, aerial guideways, cut-and-cover subway structures, tunnels, passenger stations, earth-retaining structures, surface buildings, miscellaneous structures such as culverts, sound walls, and equipment enclosures, and other non structural and operationally critical components and facilities supported on or inside Metro structures. These criteria also establish the design parameters for temporary structures. The design life objective for permanent underground structures designed to meet this criteria shall be 100 years. For aerial structures, bridges, and other structures conforming to Caltrans Bridge Design Specification (BDS) LRFD, the design life is 100 years.

These criteria also apply to existing proximate facilities and their foundations not owned by Metro, but that fall into the zone of influence of Metro's temporary and permanent facilities being designed. Where cases of special designs are encountered that are not specifically covered by these criteria, the designer shall bring them to the attention of Metro to determine the technical source for the design criteria to be used.

The main reference document controlling the seismic design of Metro facilities under these criteria is Section 5 Appendix, Metro Supplemental Seismic Design Criteria.

5.1.2 Reference Data

For the structural design, meet all applicable portions of the State of California general laws and regulations, and the codes, manuals, or specifications identified in this Section. Where the requirements stipulated in any such document or by these criteria are in conflict, use the stricter, unless otherwise noted herein.

Unless specifically noted otherwise in these criteria, the latest edition of the code, regulation and standard that is applicable at the time the design is initiated shall be used. If a new edition or amendment to a code, regulation or standard is issued before the design is completed, the design shall conform to the new requirement(s) to the extent practical or required by the governmental agency enforcing the code, regulation or standard changed, and as agreed to by Metro.

Where there are cases of special designs encountered that are not specifically covered by these criteria, the Project Structural or Geotechnical Engineer shall bring them to the attention of Metro along with proposed criteria from standards of a recognized authority that address these special designs.

5.1.3 Reference Codes

The codes outlined below under items A, B, and C each regulate their respective seismic design criteria.

A. CAL/OSHA

For buildings, stations, elevators, escalators, lines and shafts, design to meet all California Occupational Safety and Health Act (CAL/OSHA) standards.

B. Building Codes

In the County and City of Los Angeles, apply The Los Angeles County Building Code, as applicable.

C. Other Codes, Manuals, and Specifications

1. For bridges and aerial structures that support rail transit loadings, except as otherwise noted herein, use the current Caltrans Bridge Design Specification (BDS) which implements AASHTO LRFD Bridge Design Specifications, Latest Edition, with California Amendments but with Metro specified rail transit loading. This includes applying Caltrans geotechnical investigation and design of bridge foundations. All the above is referred to throughout these criteria as "Caltrans BDS." Where Caltrans BDS is not applicable, use the most appropriate code provided below, or the Manual for Railway Engineering of the American Railway Engineering and Maintenance of Way Association, referred to as "AREMA".

2. For bridges that support highway loading, use the design requirements of the applicable jurisdiction. In the absence of such requirements, use the Caltrans BDS. The AASHTO Specifications shall be adhered to in areas where the Caltrans BDS, this criteria, and/or other contract documents are silent.

3. For underground structures used for rail transit, apply Section 5.4, Underground Guideways and Structures. In addition to these criteria, also use Metro Structural standard drawings and directive drawings for Cut and Cover Subway where applicable.

4. For structures other than guideways and bridges, and underground roof systems subject to railroad or highway loading, this code adopts the latest version of the California Building Code, California Code of Regulations, Title 24, Part 2, California Building Standards Commission, based on the International Building Code. This code and its amendments are referred to herein as the Building Code.

5. For reinforced concrete retaining walls, use the Caltrans BDS for rail transit using light and heavy passenger vehicles and highway loading, and the AREMA manual for commuter, freight, and high speed railroad loading.

6. For structural steel structures other than bridges subject to railroad or highway loading, use Specifications of the Design, Fabrication and

Erection of Structural Steel for Buildings of the American Institute of Steel Construction (AISC).

7. For timber structures other than buildings within the jurisdiction of the County of Los Angeles or the City of Los Angeles and bridges subject to railroad or highway loading, use National Design Specification for Stress-Grade Lumber and its Fastenings, as recommended by National Forest Products Association, shall apply.

8. For welded structures, use Structural Welding Code - Steel of the American Welding Society, Inc., herein after referred to as “AWS D1.1”, to design welded structures not covered by the above specifications and codes.

9. For cast-iron structures, use The Gray Iron Casting Handbook of the Gray Iron Founders Society.

10. For drilled piers and caissons, use the ACI Suggested Design and Construction Procedures for Pier Foundations, hereinafter referred to as “ACI 336.3R”, for pile shaft design, use the Caltrans Pile Shaft Design Procedure.

11. For signs and poles, use the Caltrans BDS implemented AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals.

12. For seismic design, see Section 5 Appendix, Metro Supplemental Seismic Design Criteria.

D. *Project Structural Engineer* is defined herein by procurement method.

1. Design-build (D-B): Design-builder’s engineer of record’s lead structural engineer who shall be a licensed professional structural engineer as defined by the State of California Department of Commerce and Consumer Affairs (DCCA) and who shall be responsible in charge of all structural work and who shall affix his stamp and seal on all project design work. All work shall be subject to Metro review and acceptance.

2. Design-bid-build (D-B-B): Lead structural engineer who shall be a licensed professional structural engineer as defined by DCCA and who shall affix his stamp and seal on all project design work prepared for Metro either directly or indirectly as an employee of the engineer of record or as a subconsultant to the engineer of record. All design work shall be subject to Metro review and approval.

E. All structural calculations provided in support of these criteria shall be sealed by a professional structural engineer.

5.2 LOADS AND CONDITIONS

There are no exact lines that divide the design criteria and specifications between aerial guideways and their foundations, underground facilities and their surface level elements, and at-grade guideways and their associated appurtenances. Further problems occur when passenger stations and pedestrian access and egress elements are contiguous with facilities supporting guideway forces and deflections.

Nevertheless, because of the codes in current usage, it is necessary to identify the general differences between aerial, underground, and at-grade rail facilities. Where the criteria for one type of structure is not specifically applicable or available, the designer should refer to the other categories for minimum structural and geotechnical requirements.

5.2.1 General

Design all rail transit structures to sustain the maximum dead and live loads to which they may be subject, including erection loads occurring during construction and the following other loads and forces:

A. Dead loads of structural components and nonstructural attachments (DC)
B. Superimposed dead loads (DW)
C. Live loads (LL)
 1. Weight of Heavy Rail Vehicle (HRV)
 2. Weight of Heavy Rail Crane Car (HP)
 3. Weight of Light Rail Vehicle (LRV)
 4. Weight of Light Rail Maintenance Car (LP)
D. Pedestrian live load (PL)
E. Derailment loads (DR)
F. Earthquake loads (EQ)
G. Friction force (FR)
H. Dynamic load allowance (IMV, IMH)
I. Centrifugal force (CE)
J. Longitudinal force (LF)
K. Earth pressure (EH)
L. Vertical pressure from dead load of earth fill (EV)
M. Live load surcharge (LS)
N. Downdrag force (DD)
O. Earth Surcharge Force (ES)
P. Water load, steam pressure, buoyancy, scour (WA)
Q. Wind load on structure (WS)
R. Wind load on live load (WL)
S. Force effects due to shrinkage (SH)
T. Force effects due to creep (CR)
U. Locked-in construction forces (EL)
V. Secondary forces from post-tensioning (PS)
W. Force effects due to uniform temperature (TU, TTR, TLR)
X. Rail fracture (RF)
Y. Force effects due to temperature gradient (TG)
Z. Force effects due to settlement (SE)
AA. Vehicular collision loads (CT)

BB. Railroad or Vessel collision load (CV)

Other Acronyms frequently used by these criteria include:

GPR = Geotechnical Planning Report
lbs = pounds
plf = pounds per lineal foot
psf = pounds per square foot
psi = pounds per square inch
kips = kilo-pounds (1000 pounds)
ksf = kips per square foot
ksi = kips per square inch
min = minute or minimum
sq ft = square foot
cu ft = cubic foot
SPT = standard penetration test
CPT = cone resistance
Δ or δ = deflection of difference
μ = Poisson's ratio
p = pressure
γ_p = Load factor for permanent loads (See Tables 5-2, 5-3, and 5-5)
γ_{TG} = Load factor for temperature gradients (See Tables 5-2, and 5-5)
γ_{SE} = Load factor for permanent settlement (See Tables 5-2, 5-3, and 5-5)
E_S = Young's modulus of elasticity
k_S = Subgrade reaction in pounds per cubic inch
q_U = Unconfined compressive strength
q_C = CPT cone resistance
N = SPT blow count
I_P = Plasticity index

Loading criteria to which the structures are designed shall be shown on the designer's structural drawings. Concrete placing and construction sequence shall be shown on the Designer's plans when required by design conditions.

Provisions of agreements with property owners and other agencies regarding special loading for portions of structures that pass beneath or adjacent to their properties or facilities shall be considered in establishing the loading conditions for such structures. Attention shall be paid to proposed future construction.

Temporary and Staged Construction:

A. The design of all segmental girder aerial structures: The construction forces resulting from the use of an erection gantry and locked-in forces.

B. Construction Loads shall be considered in the design in accordance with Caltrans BDS implemented AASHTO LRFD Section 5.14.2.

5.2.2 Dead Loads (DC, DW)

Dead loads consist of the actual weight of the structure, permanently installed trackwork, partitions, service walks, pipes, conduits, cables, utilities, services, and all other permanent construction and fixtures. Component dead load (DC) shall consist of the weight of all components of the structure. Superimposed dead load (DW) shall include the weights of all appurtenances and utilities attached to the structure, including, but not necessarily limited to, the weights of the running rails, rail fasteners, concrete rail support (plinth) pads, emergency guardrails, catenary system and structural support, contact rail and coverboard with mountings and support pads, walkways, wireways, cable trays, cables, railings and acoustical barriers. Dead loads for all elements shall account for deck camber, curvature and superelevation. Since dead load stresses are always present, the structure must be designed to sustain them at all times without reductions.

The approximate unit weights of materials normally used in construction are shown in Table 5-1. A specific check should be made as to the actual weight where a variation might affect the adequacy of the design or where the construction may vary from the normal practice.

A. For design of aerial guideways, unit weights and loads specified in Subsection 3.5.1 of AASHTO LRFD Bridge Design Specifications shall be used. The dead load for all other structures shall be computed from the weights of the materials composing the structure and its permanent fixtures.

B. Structures Constructed by Cut-and-Cover Methods:

1. The dead load for structures constructed by cut-and-cover methods consists of the weight of the basic structure, the weight of secondary elements permanently supported by the structure, and the weight of the earth cover supported by the top of the structure and acting as a simple gravity load.

2. Apply the dead load in stages to realistically represent the lift history of the designed structure. For example, removal of the earth cover from a prestressed concrete span at some future date may create a serious upward deflection problem and should, therefore, be ~~analyzed~~ **analysed** as a separate loading case.

3. Use a design unit weight of earth, both above and below the groundwater table, of not less than 130 pcf for the analysis of the structural frame unless specified otherwise. In making calculations with regard to dead weight resisting flotation of the structure, the actual unit weight of backfill placed over the structure shall be used, but in no case shall be taken as greater than 120 pcf. Where full hydrostatic pressure below the groundwater table is used as a design load, use a submerged design unit weight of not more than 68 pcf for earth below the groundwater table.

4. Cut and cover entrance structures shall be designed for a minimum of 8 feet ground cover.

C. Loads from Adjacent Existing Building Foundations or Other Existing Structures

1. Determine the need for all permanent underpinning of buildings or structures. The underground structures shall be designed for loading from existing adjacent buildings or structures. Consideration shall be given to the maximum and minimum loads that can be transferred to the design structure, and design loads shall be assumed to be the same as those for which the adjacent structure was designed; but, in the absence

of this information, base loads on provisions in the applicable building code or the actual weights and the heaviest occupancy for which the building is suitable.

2. Horizontal and vertical distribution of loads from foundations of existing buildings shall be determined by the designer in consultation with its geotechnical engineer.

D. Miscellaneous Loads

1. Consider provisions of agreements with property owners, railroads, and other agencies regarding special loading for portions of structures that pass beneath or adjacent to their properties or facilities in establishing the loading conditions for such structures. Attention shall be paid to proposed future construction.

2. Design all aerial structures and bridges for possible future attachment of sound walls. Use the dead load for sound walls of 300 pounds per linear foot of structure per wall. Consider walls to occupy either side of the structure or both sides simultaneously.

5.2.3 Live Loads (LL, PL, LS)

A. Heavy Rail Vehicle (HRV)

Car dimensions and weights of Heavy Rail Vehicle (HRV) and Heavy Rail Crane Car (HP) are shown on Fig. 5-1.

Metro Rail Projects utilizing heavy rail technology shall normally operate trains consisting of 1, 2, or 3 dependent pair of cars. Normally, each dependent pair of heavy rail cars `are approximately 150 feet in length and are equipped with 4 trucks consisting of a total of 8 axles.

Under some abnormal operating conditions, one train will be used to push or pull a failed train. This procedure will result in two of the maximum length trains being operated over a portion of a line. Both trains may be carrying passenger loads until the next station is reached before passengers on board the failed train can be off-loaded.

Structures subjected to train loads shall be designed for any combination of train lengths, loads and forces which produce the most critical condition.

B. Light Rail Vehicle (LRV) with catenary power supply

Car dimensions and weights of Light Rail Vehicle (LRV) and Light Rail Maintenance Vehicle (LP) are shown on Fig. 5-2.

Metro Rail Projects utilizing light rail technology normally operate trains consisting of 1, 2, or 3 articulated cars. Normally, the light rail cars are approximately 90 feet in length and are equipped with 3 trucks consisting of a total of 6 axles. The number of vehicles considered in a train shall vary from 1 to the number required to add up to approximately 270 feet in total train length.

The FTA Transit Cooperative Report Program 57 designates vehicle design live loading as AW0, AW1, AW2, AW3, and AW4:

- AW0 is the total revenue service ready dead weight;
- AW1 is AW0 plus all seated passengers at 155 pounds each;
- AW2 (design load) is AW1 plus standing passengers at 4 standees per square meter;
- AW3 (crush load) is AW1 plus standing passengers at 6 standees per square meter;
- AW4 (vehicle structure design) is AW1 plus standing passengers at 8 standees per square meter.

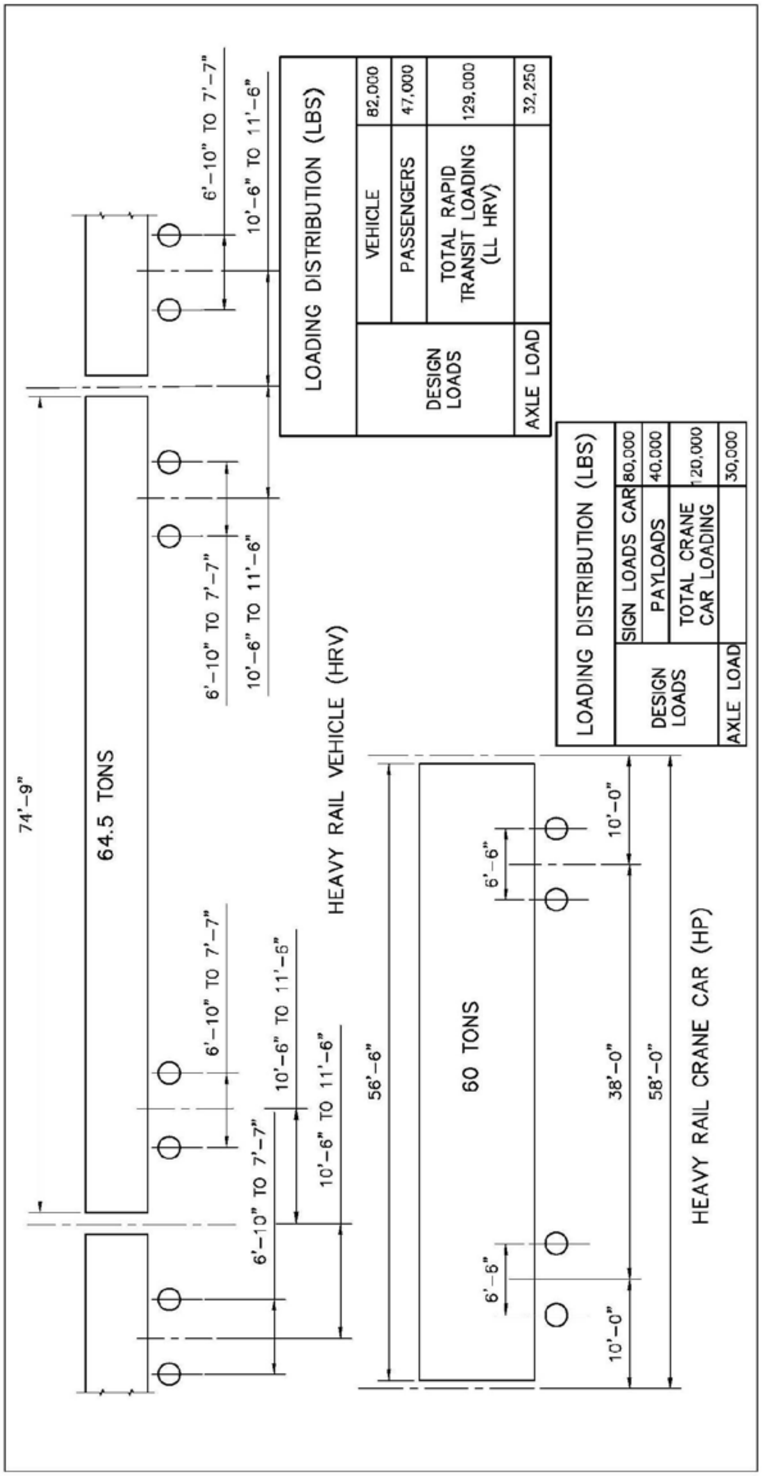
AW3 shall be used for live loads designated as LL_{LRV} in Table 5-2 for initial design, recognizing that structural calculations will be required to confirm the adequacy of the final design after the vehicle characteristics are confirmed. AW4 shall be used for live loads in Table 5-2 using the same load factors as those used for LL_{LP}. In all cases, the combination of train lengths used for structural design shall be the one that produces the most severe conditions on the element being designed.

Table 5-1 Weights of Materials

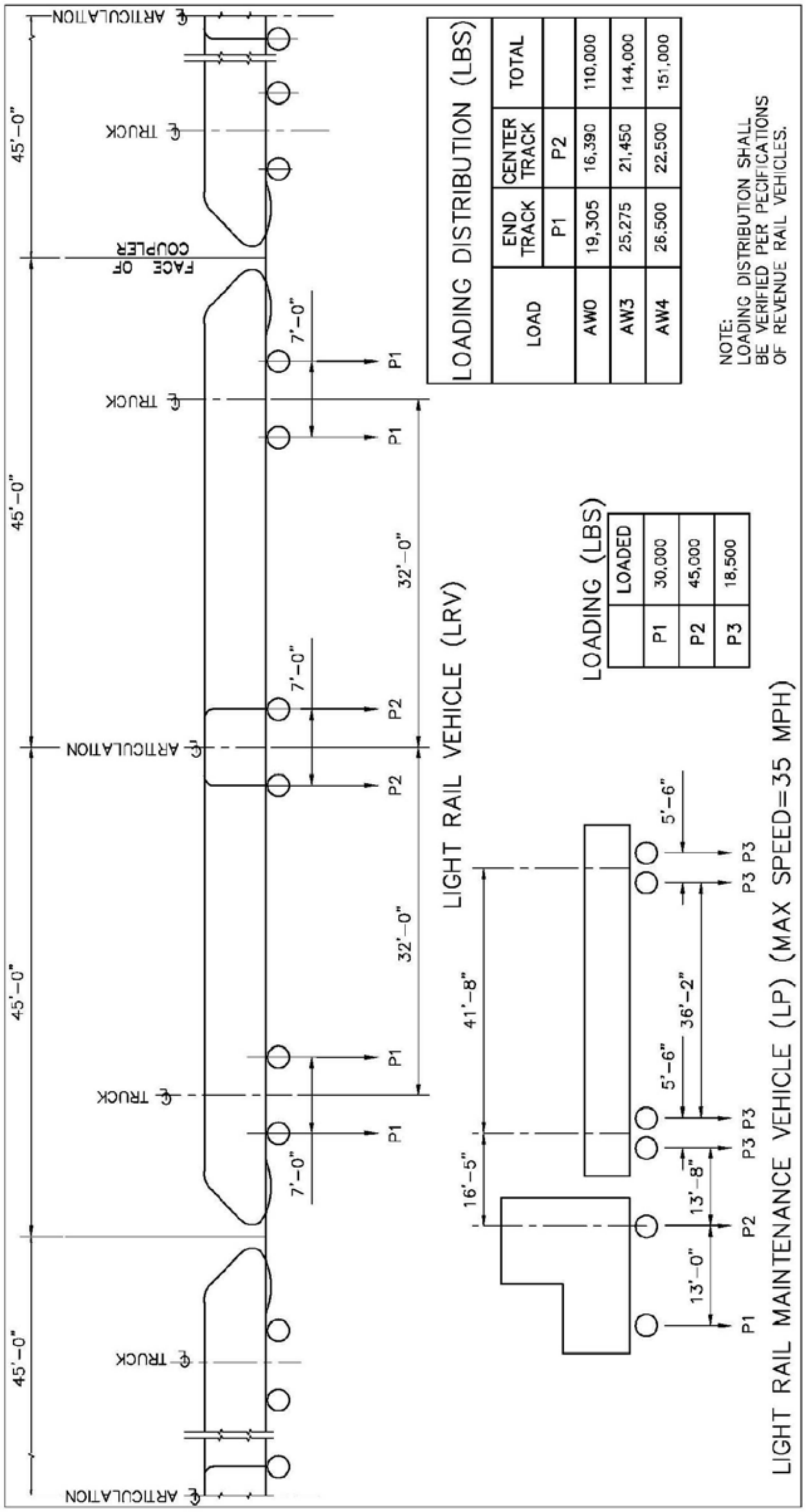
Material	Weight
Ceilings:	
Ceilings, plaster board, unplastered	3 psf
Gypsum ceiling tile, 2" unplastered	9 psf
Floors	
Concrete: plain or reinforced; gravel aggregates	150 pcf
Special and lightweight concretes	110 pcf
Gypsum floor slab, per inch of depth	5 psf
Asphalt mastic	5 psf
Ceramic tile, on 1" mortar bed	23 psf
Terrazzo, 1" on 1/2" mortar bed	18 psf
Marble, 1" on 1/2" mortar bed	20 psf
Linoleum	2 psf
Roofs	
Roofs: roofing felt, 3 ply, and gravel	5-1/2 psf
5 ply	6-1/2 psf
Maple, 7/8" on sheathing, 2" cinder fill, no ceiling	18 psf
Oak, 7/8" on sheathing, wood joists at 16" centers, no ceiling	11 psf
Walls	
Ceramic glazed structural facing tile, 4"	33 psf
Glass	160 pcf
Partitions: plaster, 2" channel stud, metal lath	20 psf
Plaster, 4" channel stud, metal lath	32 psf
Hollow plaster, 4” metal lath	22 psf
Gypsum block solid, 3" -- both sides plastered	19 psf
Gypsum block, hollow, 5"	22 psf
Marble wainscoting, 1"	15 psf
Steel partitions	4 psf
Ceramic glazed structural tile, 4"	33 psf
Sheathing, 3/4" thick	3-1/2 psf
Walls: brick solid, per inch	10 psf
Terra cotta tile 4" -- plastering, add 5 psf per side	25 psf
8" tile	33 psf
12" tile	45 psf
Glass, structural, per inch	15 psf
Windows, frame, glass, sash	8 psf
Stone, 4"	55 psf
Steel sheeting, 14 gauge	3 psf
Miscellaneous	
Timber: untreated	48 pcf
Timber: treated	60 pcf
Steel	490 pcf
Iron, cast	450 pcf
Third rail	32 plf
Rails and Fastenings, per track (2 rails)	130 plf
Gravel, sand	120 pcf
Pressed steel	2 psf
Aluminum alloy	175 pcf

Material	**Weight**
Asphalt mastic, bituminous macadam	150 pcf
Ballast, crushed stone, compacted earth	120 pcf
Overhead Contact Support System	Provided by Designer

pcf = pounds per cubic foot
psf = pounds per square foot
plf = pounds per linear foot



HEAVY RAIL VEHICLE (HRV) and HEAVY RAIL CRANE CAR (HP)
Figure 5-1



LOADING DISTRIBUTION (LBS)

LOAD	END TRACK P1	CENTER TRACK P2	TOTAL
AW0	19,305	16,390	110,000
AW3	25,275	21,450	144,000
AW4	26,500	22,500	151,000

LOADING (LBS)

	LOADED
P1	30,000
P2	45,000
P3	18,500

LIGHT RAIL VEHICLE (LRV) and LIGHT RAIL MAINTENANCE VEHICLE (LP)
Figure 5-2

C. Roadway

Base roadway LL for underground rail transit structures shall be Caltrans BDS.

Superimposed wheel load from this loading shall be distributed in accordance with the AASHTO LRFD BDS Specifications, Article 3.6.1.3.3.

The design of all underground structures shall take into account the possibility of loads due to future fills, roadways, surface treatments, and building construction.

D. Railway

Railway LL shall use Cooper E 80 loads as specified in AREMA, Chapter 8, Part 1, Section 2.2.3(c) unless otherwise specified by the railroad company.

E. Pedestrian Areas ~~(for underground stations, see Section 5.5.1)~~

1. Design station platforms, pedestrian ramps, mezzanines, and other pedestrian areas for a uniform LL of 100 psf.

2. Design stairways for a uniform LL of 100 psf or a concentrated load of 300 lb on the center of stair treads, whichever is critical.

F. Storage Space and Machinery Rooms

Design electrical equipment rooms, pump rooms, service rooms, storage space, and machinery rooms for uniform LL of 250 psf, to be increased if storage or machinery loads so dictate. Design fan rooms and battery rooms for uniform loads of 350 psf.

G. Elevators, Escalators and Passenger Conveyors

Design structures supporting elevators, escalators or passenger conveyors for the maximum reactions from any of the manufactured units considered for use in the system.

H. Railings

Design Railings in station platforms, mezzanines and service walkways for a horizontal force of 50 plf and a vertical force of 50 plf at their top.

Design railings in other places of public assembly in accordance with local codes. Design railings in equipment rooms and working areas for a force of 200 lb applied in any direction at any point.

I. Gratings

Design ventilation shaft gratings in areas that are subject to loading from vehicles to carry loading in accordance with Caltrans BDS. Design gratings in sidewalks and in areas protected from vehicular traffic for a uniform LL of 900 psf. Select types of gratings according to Article 1.16.1 and to the guidelines established by the City of Los Angeles Bureau of Engineering standard Plan S-601-2, and applicable codes listed under Section 5.1.3.

J. Service and Emergency Walks

Design service and emergency walks for a uniform LL of 85 psf of walkway area.

K. Underground Walls, Doors, and Dampers

These items are subject to air pressure from the running trains.

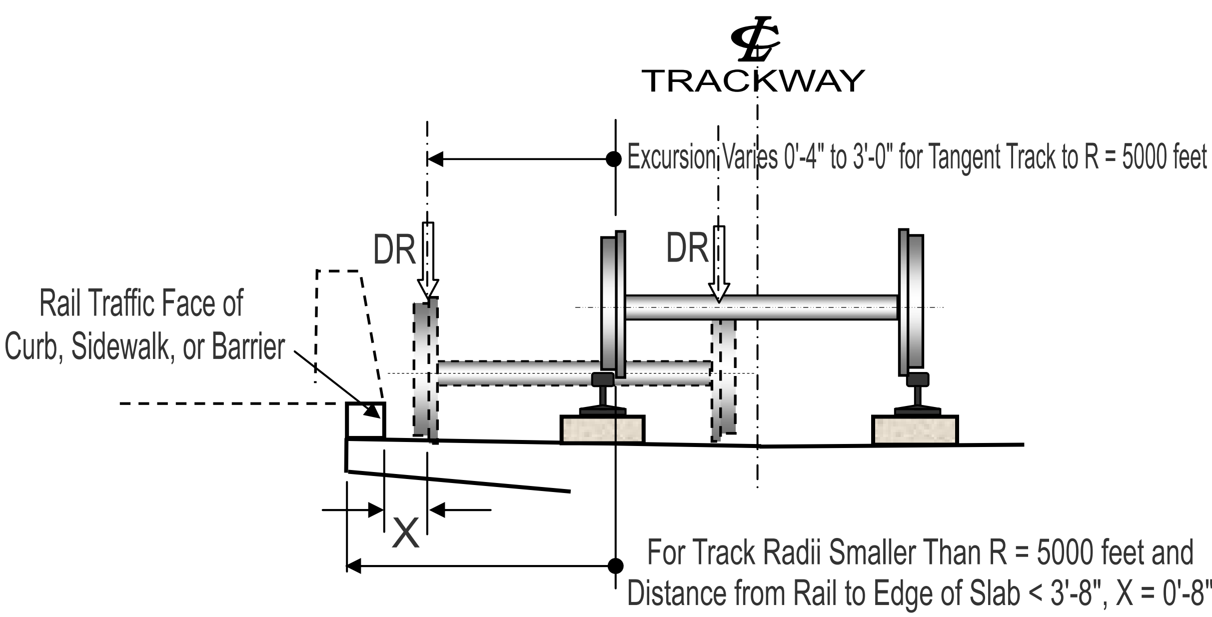
5.2.4 Derailment Loads (DR)

Guideway structures supporting HRV and LRV trains are subject to derailment forces. These shall be applied as follows.

5.2.4.1 Vertical

The vertical derailment load shall be taken as that produced by fully loaded vehicles placed with their longitudinal axes parallel to the track. Lateral vehicle excursion shall vary from 4 inch minimum to 3 feet 0 inches maximum for tangent track and curved track with radii greater than 5,000 feet. For track with smaller radii and where the distance from the rail to the edge of the deck slab is less than 3 feet 8 inches, the maximum excursion shall be adjusted so that the derailed wheel flange is located 8 inches from the rail traffic face of the nearest barrier, if any, or the edge of the deck. See Figure 5-3.

Figure 5-3 Lateral Vehicle Excursion for Vertical DR Load



A vertical impact factor of 100 percent of vehicle weight shall be used to compute the equivalent static derailment load. This vertical impact shall be in lieu of the Dynamic Load Allowance provided in the Section 5.2.6.

For derailment loads where the vehicle wheels bear directly on the slab, the wheel loads shall be assumed to be distributed over 3 feet of the slab in a direction perpendicular to the main reinforcement.

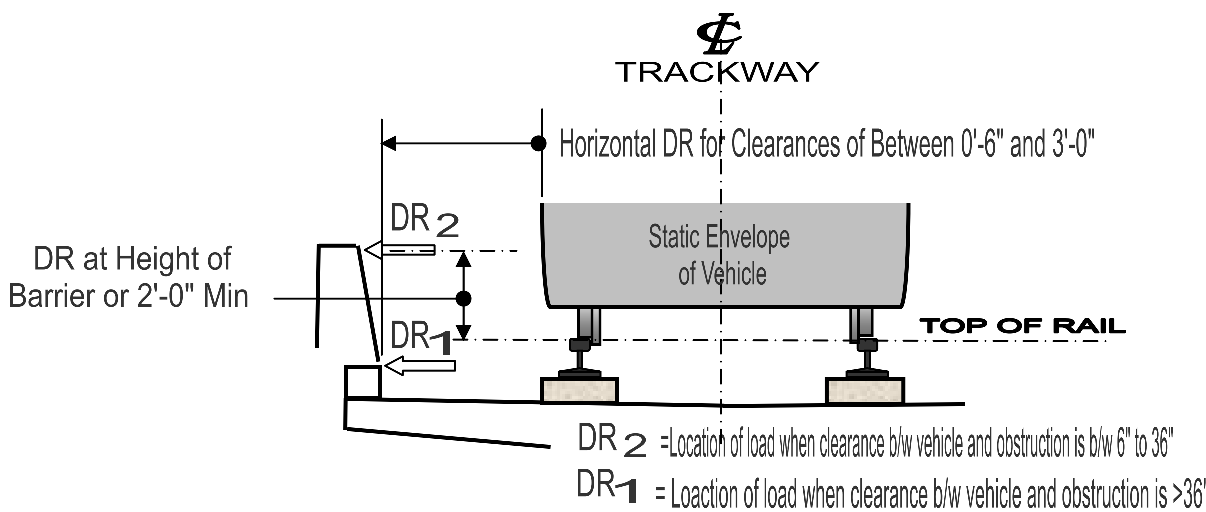
When checking any component of superstructure or substructure that supports two or more tracks, only one train on one track shall be considered to have derailed, with the other track being loaded with a stationary train without impact. All elements of the structure shall be checked assuming simultaneous application of all derailed wheel loads. However, the reduction of positive moment in continuous slabs due to derailed wheel loads in adjacent spans shall not be allowed.

5.2.4.2 Horizontal

Aerial guideways and guideways supported on embankments more than 4 feet above the surrounding grade shall be provided with restraining rails on the inside running rail on all curves on a radius of less than 500 feet. In addition, a concrete curb a minimum of 8 inches high shall be provided at the outside edge of the guideway or embankment that is above and composite with the structure supporting the guideway and structurally capable of sustaining the DR prescribed in the paragraph below.

For guideway cross-sections having a clearance between the vehicles and the barrier walls of between 6 inches and 3 feet 0 inches, with HRV and LRV speed of 55 mph or greater, the force due to horizontal DR shall be taken as 40 percent of a single fully loaded (AW3) vehicle acting 2 feet above the top of rail (DR_2) or at the top of curb (DR_1), and normal to the barrier wall for a distance of 10 feet along the wall (See Figure 5-4)

Figure 5-4 Lateral Force Distribution for Horizontal DR Load



5.2.5 Earthquake Loads (EQ)

A. All aerial structures and bridges shall be designed to resist earthquake motions in accordance with Metro Supplemental Seismic Design Criteria (Metro SSDC) Appended. In some cases, aerial structures and bridges may be under other agency jurisdictions (such as Caltrans) and design criteria specified elsewhere. Where conflict occurs, the more critical will control.

B. Underground structures and earth retaining structures subject to earthquake motions shall be designed in accordance with Metro SSDC. In some cases the nearby foundations of aerial structures and bridges may be under other agency jurisdictions (such as Caltrans) and design criteria specified elsewhere. Where conflicts occur, the more critical will control.

C. Elements of above ground station structures not subject to rail transit loading shall be designed to resist earthquake motions in accordance with the applicable building codes of Section 5.1.

D. Seismic forces for temporary and staged construction: Design response spectra shall be in accordance with Metro SSDC. Where Metro SSDC is silent on this subject, use Caltrans BDS. Seismic shall be considered in the following load combinations for staged construction:

1. For Maximum Force effects:

   Q=1.0(DC+DIFF)+1.0CE+A+EQ

2. For Minimum Force effects:

   Q=1.0DC+1.0CE+A+EQ

Where from AASHTO LRFD 5.14.2.3, Design:
A = Static weight of precast segment or element being handled (kip)

DIFF = Differential load: applicable only to balanced cantilever construction taken as 2 percent of the dead load applied to one cantilever (kip)

5.2.6 Dynamic Load Allowance (IMV, IMH)

A. Dynamic load allowance is the statically equivalent dynamic effect resulting from vertical and horizontal acceleration of the LL given as a percent of LL.

Dynamic load allowance considerations for aerial structures supporting rail transit loading shall be as follows:

1. Dynamic load allowance shall be used for the design of the superstructure and generally to those members of the structure that extend down to the main footings as well as the portion above the ground line of concrete or steel piles rigidly connected to the superstructure. Dynamic load allowance shall not be used for abutments, retaining walls, wall-type piers, embedded piles, footings, and service walks.

2. Vertical dynamic load allowance (IMV) for aerial structures shall be 33 percent of LL.

3. In addition to IMV provided above, a horizontal dynamic load allowance (IMH) equal to 10 percent of LL shall be applied. This force shall be equally distributed to the individual axles of the vehicle and shall be assumed to act in either direction transverse to the track through a point at 3.5 feet above the top of the low rail.

   The horizontal force component transmitted to the rails and supporting structure by an axle shall be concentrated at the rail having direct wheel flange to rail head contact. When IMH acts simultaneously with CE, only the larger of the two forces needs to be considered.

B. Design of the top slab of utility vaults and other underground structures supporting highway loading shall conform to the following:

IM = 33(1.0 - 0.125De) ≧ 0%

where: De = Minimum depth of earth cover above the structure (feet)

The depth of cover shall be measured from the lowest top of ground or paving to the top of the underground structure.

C. Structures supporting special vehicles, such as moving equipment or other dynamic loadings that cause significant impact, shall conform to the local building code or, if not covered by code, shall be considered individually using the best technical information available.

5.2.7 Centrifugal Force (CE)

Structures on curves shall be designed for a horizontal radial force (CE) equal to the following percentage of the LL, without Dynamic Load Allowance, in all trackways:

$$CE = f\,(V)^2 / gR$$

where: g = 32.2 feet/second^2
V = design speed (feet/second)
f = 4/3 for load combination other than fatigue and 1.0 for fatigue
R = the radius of the curve of the track centerline (feet)

The centrifugal force shall be applied 4 feet above the top of low rail on all tracks.

5.2.8 Longitudinal Force (LF) (BR)

5.2.8.1 Forces due to Acceleration and Deceleration

Provision shall be made for LF due to the train acceleration and deceleration. The magnitude of LF shall be computed as follows:

A. For decelerating trains, LF shall be equal to 28 percent of LL without dynamic load allowance. Emergency braking (BR) shall be equal to 42 percent of LL without dynamic load allowance.

B. For accelerating trains, LF shall be equal to 14 percent of LL without dynamic load allowance.

This force shall be applied to the rails and supporting structure as a uniformly distributed load over the length of the train in a horizontal plane at the top of the low rail. Consideration shall be given to various combinations of acceleration and deceleration forces where more than one track is carried by the structure except for BR that shall be applied only as a singular event.

5.2.8.2 Forces due to Restraint of Continuous Welded Rail (CWR)

Detailed calculations shall be provided to demonstrate the forces and distortions occurring at the interface between the continuous welded rail (CWR) and all elements of the supporting aerial guideway structure.

Wherever a CWR is terminated, provision shall be made to fully restrain its end. This restraint shall be assumed to introduce an LF in the end of each rail of 165,000 pounds based on 85ºF temperature change. Unless aerial structures and direct fixation bridges are designed to resist this force, CWR shall not be terminated thereon. See Trackwork Standard Drawings.

Termination, as used in the above paragraph, means absolute termination. The placement of a turnout or crossover between ends of CWR does not necessarily result in absolute termination of the rail; the CWR is not considered to be terminated if some means is provided, through the turnout or crossover, to transmit the above force from the end of one rail to the end of the other. The rail shall extend beyond the aerial or bridge structure such that a minimum of 100 rail fasteners, adjacent to each other, are engaged in the continuous at-grade or underground portions of the track.

5.2.8.3 Forces due to Rail Bumping Posts

A rail-mounted vehicle retarding device in the form of bumper posts shall be used on stub-end tracks located in yards, on main lines, or on sidings.

The transfer of loads due to collision between any number of rail transit cars, traveling at the design speed and any structure-mounted rail bumping post shall be limited to 200 kilo pounds (kips), including impact. The bumping post shall be attached only to the rail it protects and shall transfer load to the structure only through rail seat assemblies. The structure will be designed for the loads transmitted through the rail seat assemblies for only one bumping post being activated at one time.

To further protect the structure, the bumping post shall be designed with mountings so that excess loads will cause the bumper to slide over a safe distance. As an alternative to a frangible mounting, the design shall preclude any device that would cause the transferred loads to exceed 200 kips. For structural design, the bumping post load shall be evenly divided between the two rails it is attached to. Structures shall be designed to resist the lesser of 200 kips or the total available restraint provided by the rail seat assemblies on the structure supporting the rails and the bumping post in question.

5.2.9 Earth Pressures (EH, EV, ES)

A. Earth pressures shall be as specified in AASHTO LRFD Section 3.11.

B. Surcharge loads values shall not be less than those specified in AASHTO LRFD Section 3.11.6.

1. Rail transit loading shall be based on actual axial loads, including impact factor, and car spacing.

2. Vehicle [non-rail transit] loading shall be in accordance with AASHTO LRFD Section 3.11.6.

3. LL and DL from adjacent foundations of structures within the zone of influence shall be considered in computing horizontal pressures on new or existing structures. The zone of influence is defined as being a line projected downward at a slope of 1H:1V from the outside edges around the entire perimeter.

4. The lateral earth pressures to be used in design of structures either fully or partially embedded in “rock” shall be per the recommendations of the project geotechnical engineer as defined in the geotechnical section herein.

5. Earth pressures provided by the geotechnical investigation under Section 5.6, Geotechnical and Metro SSDC, take precedence if exceeding those referenced above. Also, see attached references.

5.2.10 Water Load, Stream Pressure, Buoyancy, Scour (WA)

Design ground water shall be in accordance with recommendation from the project geotechnical engineer and geotechnical data obtained from subsurface data. In addition, design surface water levels, if any, shall be in accordance with site/area-specific hydraulics report. The effects of hydrostatic pressure and buoyancy shall be considered whenever groundwater is present or may be present at a future date. The possibility of future major changes in groundwater elevation shall be considered. The design shall take into account the effect of hydrostatic pressures pertaining to construction sequence. The backfill shall be considered as the volume contained within vertical planes defined by the outside limits of the structure. Soil resistance on the sides of the structure and vertical planes defining outside limits of the structure shall be defined by the Geotechnical report.

Local flooding may add to loading on structures within the flood plain. Anticipated flood elevations shall be determined by a study of official flood records. The consequences of changes in foundation conditions resulting from the “check flood” for bridge scour and “design flood” for scour shall be considered. Water load shall be included in the design of aerial structures where applicable. All piers and other portions of structures that are subject to flood forces shall be designed in accordance with the requirements outlined in Caltrans BDS.

Guideways that cross over flood control channels and rivers shall meet requirements of the Los Angeles County Flood Control Districts and the Corps of Engineers.

5.2.11 Force Effects due to Temperature Gradient (TG)

Temperature gradient shall be considered, if applicable. Internal stresses and structural deformations due to both positive and negative temperature gradients may be determined in accordance with the provision of Caltrans BDS implemented AASHTO LRFD Section 3.12.3.

5.2.12 Force Effects due to Shrinkage and Creep (SH, CR)

Stresses and movements resulting from concrete shrinkage and creep shall be incorporated into the design of the structures in accordance with Caltrans BDS.

5.2.13 Force Effects due to Uniform Temperature (TU, TTR, TLR)

A. Provision shall be made for stresses and deformations resulting from temperature ranges as follows.

1. Concrete

   a. Temperature range = TmaxDesign - TminDesign = 60ºF (see Caltrans BDS.)

   b. Coefficient of expansion .0000060 inch/inch/ºF

2. Steel

   a. Temperature range = TmaxDesign - TminDesign = 75ºF (See Caltrans BDS).

   b. Coefficient of expansion .0000065 inch/inch/ºF

3. Direct Fixation Track

a. Controlled setting temperature

b. 80ºF minimum

c. 95ºF maximum

d. Temperature rise 34ºF maximum

e. Temperature fall 43ºF maximum

f. Coefficient of expansion 0.0000065 inch/inch/ºF

The temperature ranges specified above are based on a range of ambient air temperature of 52ºF (minimum) to 94ºF (maximum). The CWR is assumed to achieve a minimum temperature of the ambient air temperature and a maximum temperature of 20ºF above the ambient air temperature.

B. For direct fixation track, provision shall be made for transverse and longitudinal forces due to temperature variations in the rail. These forces shall be applied in a horizontal plane at the top of the low rail as follows:

1. Transverse Force (TTR): The transverse force per linear foot of structure per rail shall be determined by the following formula:

$$T = 151 \text{ Kips}/R$$

where: R = radius of curvature in feet

2. Longitudinal Force (TLR): The longitudinal force per structure per rail shall be determined by the smaller of 200 kips or by the following formula:

$$T = 0.65 \times P \times L$$

where: P = longitudinal restraint force of rail per linear foot
L = average length of adjacent structures (feet)

5.2.14 Rail Fracture (RF)

The final design of structures shall consider the possibility of any one CWR breaking under a tensile load of 200 kips. The break will be restrained by a longitudinal restraint force in the range of 1,600 pounds to 2,200 pounds per rail seat assembly. The structures will be designed for the possibility of only one rail break at one time.

Structures shall be designed to resist the lesser of 200 kips from the rail break or the total available restraint available from the rail seat assemblies on the structure for that rail. Rail seat assemblies will be spaced typically at 30 inches on-center except at bonded rail joints and at special trackwork.

At special trackwork locations, design details for anchoring rails using the same type of rail fasteners as the typical structures shall be provided.

5.2.15 Force Effects due to Settlement (SE)

Load(s) induced on the structures by differential settlement shall be considered in the loading combination. Consider this load similar to shrinkage and thermal forces or as provided in the section on settlement and deflection below. Requirements for allowable differential settlements are prescribed in the Section 5.6, Geotechnical.

5.2.16 Vehicular Collision Loads (CT)

Piers or other support elements for elevated guideways or roadways which have less than 30 feet clearance from the edge of travel way of an adjacent roadway, or less than 50 feet from the centerline of a railway track, shall be designed to withstand a horizontal static force of 400 kips, unless protected with suitable barriers. This force is assumed to act in any direction in a horizontal plane at a height of 4 feet above ground level. This condition occurs with the dead load of the structure but need not be applied concurrently with other applied loadings.

5.2.17 LRFD Design Specifications, Design Life, and Limit States

The following LRFD Design Specifications pertain mainly to aerial guideways, bridges, and structural elements being supported by these structures. Use the Caltrans BDS method for the design of all structural components and connections. Each component and connection shall satisfy each of the following limit states, unless noted otherwise in another area of these criteria.

The Project Structural Engineer is expected to use his professional judgment to determine whether or not Caltrans BDS is the applicable code for underground structure design. In general, for structures other than bridges and underground roof systems subject to railroad or highway loading, for underground structures, this code adopts the latest version of the California Building Code, California Code of Regulations, Title 24, Part 2, California Building Standards Commission, based on the International Building Code. This code and its amendments are referred to herein as the Building Code.

Applying the concepts of LRFD leads to a Caltrans BDS specified design life of 75 years. Design Life as used here means the period of time on which the statistical derivation of transient loads is based. However, with the additional seismic and other precautions taken and the mainly static forces applied to aerial and underground Metro structures, the service life for aerial and underground structures carrying rail transit as designed under these criteria is 100 years.

LRFD employs specified limit states to achieve the objectives of constructability, safety, and serviceability. A Limit State is defined as a condition beyond which a structure or structural component ceases to satisfy the provisions for which it was designed. The resistance of components and connections are determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current specifications as a result of incomplete knowledge of inelastic structural action.

LRFD uses extreme event limit states to ensure the structural survival of structures during a major earthquake or flood, or when there is a potential collision by rail or rubber tired vehicles. Extreme Event Limit States are considered to be unique occurrences whose return period may be significantly greater that the design life of the bridge.

LRFD also classifies structures on the basis of operational importance. Such classification is based on the social-survival-and/or security-defense requirements. Metro is responsible for declaring a structure or structural component to be operationally important.

5.2.17.1 Service Limit State (See Tables 5-2, 5-3, and 5-5)

A. Service I: Load Combination relating to operational use of the guideway with operational wind.

B. Service II: Load Combination intended to control yielding of steel structures and slip of slip-critical connections due to live load.

C. Service III: Load Combination for longitudinal analysis relating to tension in prestressed concrete structures with the objective of crack control and to principal tension in the webs of segmental concrete girders.

D. Service IV: Load Combination relating only to tension in prestressed concrete substructures with the objective of crack control.

E. Service V: Load Combination relating to only control of uplift and concrete tension during derailment.

F. Service VI: Load Combination relating only to segmental bridges, with no live loads and full temperature gradient.

5.2.17.2 Fatigue and Fracture Limit State (See Tables 5-2, 5-3, and 5-5)

A. Fatigue I: Fatigue and fracture load combination relating to repetitive live load and dynamic response for transit and roadway vehicles.

B. Fatigue II: Fatigue and fracture load combination relating to repetitive live load and dynamic response for transit and roadway maintenance and permit vehicles.

5.2.17.3 Strength Limit State (See Tables 5-2, 5-3, and 5-5)

A. Strength I: Load Combination relating to operational use of the guideway without wind.

B. Strength II: Load Combination relating to use of Owner-specified permit vehicles without wind.

C. Strength III: Load Combination relating to non-operational use of the guideway with high velocity wind.

D. Strength IV: Load Combination relating very high dead load to live load force effect ratios.

E. Strength V: Load Combination relating to operational use of the guideway with operational wind.

F. Strength VI: Load Combination relating to operational use of the guideway with emergency braking (BR).

5.2.17.4 Extreme Event Limit State (See Tables 5-2, 5-3, and 5-5)

A. Extreme Event I: Load Combination relating to operational use of guideway during the Maximum Design Earthquake (MDE) seismic event for connection of superstructure to substructure only (See Metro SSDC).

B. Extreme Event 1A: Load Combination relating to operational use of the guideway with the Operational Design Earthquake (ODE). See Appendices A and B.

C. Extreme Event II: Load Combination relating to operational use of guideway during a vehicle or a railroad collision (CT). (Vehicle and railroad collisions are considered to be separate events and should not be applied simultaneously. See Section 5.2.16).

D. Extreme Event III: Load Combination relating to operational use of the guideway during a derailment.

E. Extreme Event IV: Load Combination relating to a rail fracture.

5.2.18 Application of Loadings

Where applicable, use loads and forces listed above for the design of rail transit aerial structures. Rail transit vehicle live loads, buoyancy, wind loads and other variable loads shall be reduced or eliminated to create the maximum force effect on the structure. When all or a portion of deck width is dedicated exclusively to rail transit, apply only the rail transit loads to that width.

5.2.19 Multiple Presence Factor

For structures carrying rail transit loads, tracks shall be treated as a traffic lane in applying the provisions of Caltrans BDS, except the multiple presence factor for the first two loaded tracks shall be 1.0 and for three or more loaded tracks shall be 0.85.

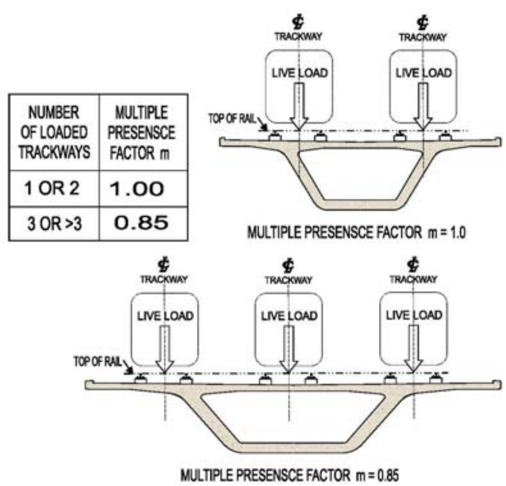


Figure 5-5 Multiple Presence Factor

5.2.20 Loading Factors and Loading Combinations, Bridges and Aerial Guideways

The following groups (Table 5-2) represent various combinations of loads and forces to which guideway and bridge structures may be subjected. (See discussion under Section 5.2.17). Each structural component shall be designed for the appropriate load combination limit states and load factors as specified in Caltrans BDS. Additionally, for precast segmentally constructed bridges, consider load combination in Caltrans BDS implemented AASHTO LRFD equation 3.4.1-2 for service limit state (Service VI in Table 5-2).

Table 5-2 Loading Combinations and Load Factors, Bridges and Aerial Guideways

Load Combination Limit State	Permanent Loads		Transient Loads						Loads Due to Volumetric Change		Exceptional Loads Use One of These at a Time					
	DC DD DW EH EV ES EL PS CR SH	SE	LL LL_{HL93} LL_{HRV} LL_{LRV} IMV IMH CE LF/BR PL LS	LL_{PERMIT} LL_{HP} LL_{LP} IM CE	WA	WS	WL	FR	TU** TTR** TUR**	TG	EQ	CT	CV	DR	RF
Strength I	γ_p	γ_{SE}	1.75	--	1.00	--	--	1.00	--	--	--	--	--	--	--
Strength II	γ_p	γ_{SE}	--	1.35	1.00	--	--	1.00	--	--	--	--	--	--	--
Strength III	γ_p	γ_{SE}	--	--	1.00	1.40	--	1.00	--	--	--	--	--	--	--
Strength IV	γ_p	--	--	--	1.00	--	--	1.00	--	--	--	--	--	--	--
Strength V	γ_p	γ_{SE}	1.35	--	1.00	0.40	1.40	1.00	--	--	--	--	--	--	--
Strength VI	γ_p	γ_{SE}	1.35	--	1.00	--	--	1.00	--	--	--	--	--	--	--
Extreme Event I	1.00	--	1.0*	--	1.00	--	--	1.00	--	--	1.00	--	--	--	--
Extreme Event IA	1.00	--	1.0*	--	1.00	--	--	1.00	--	--	1.00	--	--	--	--
Extreme Event II	1.00	--	1.0*	--	1.00	--	--	1.00	--	--	--	1.00	1.00	--	--
Extreme Event III	γ_p	γ_{SE}	1.0	--	1.00	--	--	1.00	--	--	--	--	--	1.00	--
Extreme Event IV	γ_p	γ_{SE}	--	--	--	--	--	1.00	--	--	--	--	--	--	1.30
Service I	1.00	γ_{SE}	1.00	--	1.00	0.30	1.00	1.00	--	γ_{TG}	--	--	--	--	--
Service II	1.00	--	1.30	--	1.00	--	--	1.00	--	--	--	--	--	--	--
Service III	1.00	γ_{SE}	0.8	--	1.00	--	--	1.00	--	γ_{TG}	--	--	--	--	--
Service IV	1.00	1.00	--	--	1.00	0.70	--	1.00	--	--	--	--	--	--	--
Service V	1.00	1.00	--	--	1.00	1.00	--	1.00	--	γ_{TG}	--	--	--	--	--
Service VI	1.00	--	--	--	1.00	--	--	1.00	--	γ_{TG}	--	--	--	--	--
Fatigue I LL_{HL93}, LL_{HRV}, LL_{LRV}, IMV, IMH, & CE Only	--	--	0.75	.075	--	--	--	--	--	--	--	--	--	--	--
Fatigue II LL_{PERMIT}, LL_{HP1}, LL_{LP}, & IM	--	--	--	1.00	--	--	--	--	--	--	--	--	--	--	--

* Live load from Heavy Rail Vehicle (HRV) and Light Rail Vehicle (LRV) to be loaded on one track only.

** Larger value shall be used for deformations and smaller value for all other effects.

γ_p Values, See Caltrans BDS referenced AASHTO Table 3.4.1-2 and Table 3.4.1-3, Load Factors for Permanent Loads, except as noted herein.

γ_p Values for PS, CR, and SH; See Caltrans BDS referenced AASHTO Table 3.4.1-3, Load Factors for Permanent Loads Due to Superimposed Deformations.

γ_p Values for EL shall equal the value for DC.

γ_{TG} Values for Service I, III, V, & VI shall be 0.5.

γ_{SE} The load factors for settlement should be considered on a project-specific basis in accordance with the GPR. See Section 5.2.15.

5.2.21 Load Distribution

Distribute live loads in accordance with provisions of Caltrans BDS, except as noted herein. Modify Caltrans BDS by the following additions:

5.2.21.1 Ballasted Track

Axle loads may be assumed as uniformly distributed longitudinally over a length of 3 feet, plus the depth of ballast under the tie, plus twice the effective depth of slab, except as limited by axle spacing.

Wheel loads may be assumed to have uniform lateral distribution over a width equal to the length of the tie plus the depth of ballast under the tie, except as limited by the proximity of adjacent tracks or the extent of the structure.

5.2.21.2 Direct Fixation Track

Where wheel loads are transmitted to the deck slab through rail mountings placed directly on the slab, the wheel load shall be assumed as uniformly distributed over a length of 3 feet along the rail. This load may be distributed transversely (normal to the rail and centered on the rail) by the width of the rail fastener pad plus twice the depth of the deck and track concrete.

5.3 AERIAL GUIDEWAYS AND STRUCTURES

5.3.1 Wind Load on Structure (WS)

For structures other than guideways and bridges subject to wind loading, See Section 5.1.3.C.4.

The aerial structures shall be designed to withstand wind loads of uniform pressure acting upon the superstructure, substructure, and live load (see the wind load on live load section below).

5.3.1.1 Wind Load on Superstructure

A horizontal uniform wind load of the intensities given by Caltrans BDS implemented AASHTO LRFD Section 3.8.1.2.2 shall be applied simultaneously at the centroid of all exposed areas. In addition to the horizontal wind loads, an upward load shall be applied at the windward quarter point of the transverse width of the superstructure. This vertical load shall be as specified in AASHTO LRFD Section 3.8.2. Wind loading on catenary shall be considered in the design of both the superstructure and substructure elements. Loads (magnitude and location) shall be determined by the OCS consultant, but shall consider the forces specified by AASHTO LRFD Section 3.8.3.

5.3.1.2 Wind Load on Substructure

The substructure shall be designed to withstand the preceding loads applied to the superstructure as they are transmitted to the substructure. In addition, a horizontal wind load of magnitude specified in Caltrans BDS implemented AASHTO LRFD Section 3.8.1.2.3 in any direction shall be applied simultaneously at the centroid of the exposed projected substructure area.

5.3.2 Wind Load on Live Load (WL)

A. For trains operating on aerial structures with the underside of the main girders not more than 40 feet above the mean retarding surface, WL shall consist of a transverse wind load of 115 plf of train and a longitudinal wind load of 28 plf of train. These loads shall be applied simultaneously. The transverse force shall be applied to the rail and superstructure as loads concentrated at the axle locations and in plane 6 feet 4 inches above the top of the lower rail. The longitudinal force shall be applied to the rails and superstructure as a load uniformly distributed over the length of the train in a horizontal plane 6 feet 4 inches above the top of the lower rail.

B. For higher aerial structures, the values of WL in the transverse and longitudinal directions shall be as follows:

H = 41 feet to 60 feet

where: Transverse wind pressure = 126 plf
Longitudinal wind pressure = 31 plf

H = 61 feet to 100 feet

where: Transverse wind pressure = 130 plf
Longitudinal wind pressure = 34 plf

Where H is measured from the mean retarding surface to the underside of the main girder.

These WL loads also apply to the design of substructure elements supporting a single track, or the design of substructure elements supporting two tracks. WL loads on a single train shall be increased by 30 percent when both tracks are loaded; this factor accounts fully for shielding effect of vehicle-on-vehicle as the two trains run alongside each other.

5.3.3 Special Design Considerations

5.3.3.1 Vertical Vibration

A moving vehicle exerts a dynamic effect on the guideway resulting from a highly complex interaction of the vehicle suspension system, vehicle speed, and roughness of the riding surface with the guideway. In order to avoid resonance and provide passenger comfort, an analysis of the dynamic interaction between the vehicles and the guideway structure shall be performed.

To limit vibration amplification due to the dynamic interaction between the superstructure and the rail car(s), the first-mode natural frequency of vertical vibration of each simple span guideway should generally be not less than 2.5 hertz and no more than one span in a series of three consecutive spans should have a first-mode natural frequency of less than 3.0 hertz.

Special analysis shall be performed for any bridge or for superstructures having a first mode of vertical vibration less than 2.5 hertz or for the condition when more than one span in a series of three consecutive spans has the first mode of vibration less than 3.0 hertz.

This special analysis shall model the proposed structure and the transit vehicle. The analysis shall contain a sufficient number of degrees of freedom to allow modeling of the structure, vehicle truck spacing, vehicle primary suspension, vehicle secondary suspension, and the car body. It shall make provision for the placement of the vehicle on the structure in various locations to model the passage of the transit vehicle. When the exact configuration of either the vehicle or the structure is not known, the analysis shall assume a reasonable range of parameters and shall model combinations of those parameters as deemed appropriate.

The analysis shall determine whether vertical dynamic load allowance loads in excess of 33 percent of LL are required for the design of the structure.

Thermal force interaction between the structural components and the trackwork system shall be considered, as specified in the section on force effects due to uniform temperature above.

5.3.3.2 Fatigue

The effect of stress level changes caused by passage of rail trains over structures shall be considered using 3 million cycles of maximum stress over the life of the structure.

5.3.3.3 Uplift

There should be no uplift at any support for any combination of loading. See the section on loading combinations herein.

5.3.3.4 Friction

Friction shall be considered in the design where applicable.

5.3.3.5 Sound Barriers

Sound barriers, both presence and absence, shall be considered in the evaluation of stress, vibration, and deflection limits.

5.3.3.6 Bearings

Caltrans BDS shall be used for design of bearings.

5.3.3.7 Camber Growth and Deflections for Aerial Guideway Structures

As a guide in design, the total long-term predicted camber growth, less deflection due to full dead load, shall be limited to 1/2000 of the span length for non-ballasted, prestressed concrete aerial structures, unless approved otherwise by Metro.

To ensure rider comfort, the deflection of longitudinal girders under normal live load plus dynamic load allowance shall not exceed 1/1000 of the span length. For main cantilever girders, the deflection under normal live load with dynamic load allowance shall not exceed 1/375 of the cantilever span.

The differential deflection of the slab immediately below the centerline of the two rails of the same track, due to girder and slab deformations, shall not exceed 1/5000 of the span length.

5.3.3.8 Longitudinal Tension Stresses in Prestressed Members

Caltrans BDS implemented AASHTO LRFD Bridge Design Specifications, and California Amendments shall be used for allowable longitudinal tension stresses. Tension stresses are not allowed in pre-compressed tensile zones after all losses have occurred.

5.3.3.9 Structure Deformations and Settlements

The control of deformations through proper structural design is of paramount importance in obtaining acceptable ride quality for the transit vehicles and passengers. Consider all structure deformations, including foundation settlement, not only for their effects on structural behavior but also for their effect on trackwork. As a minimum, guideway piers and abutments settlement as measured at the top of concrete of the finished guideway girder deck shall be limited as prescribed in the section on settlement and deflection below.

5.3.3.10 Additional Requirements for Precast Segmental Guideway Construction

A. Shear and torsion design to conform to Caltrans BDS implemented AASHTO LRFD Section 5.8.6.

B. Principal tensile stresses in webs to conform to AASHTO LRFD Section 5.8.5.

C. If precast columns are used, the columns shall have access opening for future inspection. The columns shall have a solid section minimum 5 feet above finished grade or 12 feet above high water level. Vertical Post-tensioning is not allowed in the solid sections.

D. Dry joints not allowed in the superstructure and substructure precast elements with match cast joints.

E. Box girders shall be transversely post-tensioned. No transverse pre-tensioning is allowed.

5.3.3.11 Crack Control

The design of prestressed concrete aerial structures shall consider the effect of temporary loads imposed by sequence of construction stages, forming, falsework, and construction equipment, as well as the stresses created by lifting or placing pre-cast members, stress concentration (non-uniform bearing at the ends of pre-cast beams), end block design and detailing, methods of erection, shrinkage, and curing. Ensure that the structural design of all pre-stressed or reinforced concrete members is adequate and clear and that specifications are prepared which are compatible with the design so that objectionable cracking does not occur in erection or service.

5.3.3.12 Special Structures

Some special structures and structural systems involve unique design and construction problems not covered by these criteria. Retrofit repairs, alterations and additions necessary for the preservation and restoration of historic buildings, bridges, and structures may be made without strict conformance to these criteria when authorized by Metro. See also, Chapter 3, Part A.

5.3.4 Vibration Criteria for Structures Supporting Pedestrian Traffic Only

To avoid the possibility of resonant vibrations induced by pedestrian traffic, the natural frequency of the unloaded structure shall be not less than 3.0 hertz. To avoid vibrations that might be objectionable to patrons, the calculated live load deflection shall be limited to 1/500 of the span length.

5.3.5 Seismic Design for Structures Supporting Pedestrian Traffic Only

Station platforms, pedestrian ramps, pedestrian bridges, and mezzanines shall be designed to resist earthquake motions in accordance with Metro Supplemental Seismic Design Criteria (Metro SSDC). In some cases, these structures may be under other agency jurisdictions and shall be designed to resist earthquake motions in accordance with the applicable Building Code or Caltrans BDS, whichever is stricter. Refer to Section 5.1 for more detailed information.

5.3.6 Material Design Requirements and Criteria

5.3.6.1 Reinforced Concrete Design

A. Minimum material properties: For all above ground reinforced concrete cast-in place structures, including columns, cap beams, and superstructure for aerial structures and bridges, columns, beams, slabs, foundations, and walls for the buildings:

f'c = 4000 psi minimum.

B. For all cast in place drilled shaft foundations:

f'c = 4500 psi minimum.

1. Mix design shall account for construction method, reinforcement clear space openings, and estimated time of placement.
2. Maximum 3/8-inch aggregate shall be used and rebar minimum clear spacing 5 inches unless it is demonstrated that drilled shaft reinforcing cage clear space opening of at least 10 times the maximum size aggregate is maintained.
3. No accelerants shall be permitted.
4. Temperature monitoring of trial and test shafts shall be performed at three locations within the shafts to establish heat of hydration development within the as-placed shaft trial mix concrete. The data acquisition system shall be capable of acquiring, storing, printing, and downloading [archiving] data to a computer. Temperature sensors shall be in the upper 20 feet and top and bottom of the middle third as measured along the length of the shaft. For purposes of temperature monitoring, the shaft diameter groupings shall be:
   a. Under 8 feet
   b. 8 feet to 10 feet, inclusive
   c. 10 feet or greater up 14 feet
   d. Greater than 14 feet
5. Type-IV or Type-II (moderate heat) cement may be used in lieu of temperature monitoring.
6. Supplementary cementitious materials if used shall be fly ash, blast furnace slag, and natural pozzolan, excepting Class-C fly ash, which is prohibited.

7. Mix design shall address the workability requirements for drilled shaft concrete over a period of time exceeding expected duration of the pour. Workability of shaft concrete shall be ensured over the expected duration of pours such that slump measured at expected duration of pour plus 2 hours shall not be less than 6 inches. Duration of estimated pours shall take into account travel and any stand-by times and be based on substantiated placement production rates.

8. Once a mix design has been approved, it shall not be changed without substantiation as described above.

C. For prestressed concrete:

f'c = 6000 psi minimum.

D. For all building foundations, floor slabs, pits, and other miscellaneous foundations at yards and shops; miscellaneous foundations other than those specified; and station platform foundations:

f'c = 3000 psi minimum.

E. In certain cases, strengths of concrete other than those specified above might be required. These cases will be as recommended by the Designer and accepted by Metro.

F. Reinforcing steel: Bar reinforcement shall conform to AASHTO M 31 for billet-steel bars or ASTM A706 for low-alloy steel bars and the following requirements:

1. Bars shall be deformed type.

2. Bars shall be Grade 60 or, for ASTM A706 bars or when specified for AASHTO M 31 bars, Grade 60.

G. Prestressing steel: Stress relieved steel strand ASTM A416 (AASHTO M 203) (low relaxation), high strength steel bar ASTM A722 (AASHTO M 275).

5.3.6.2 Structural Steel Design

A. Structural steel channels, angles, MC shapes: ASTM A36 or ASTM A50.

B. Structural steel W shapes for building frame: ASTM A992.

C. Structural steel tube: ASTM A500 Gr B.

D. Structural steel pipe: ASTM A53 Gr B.

E. For uses requiring higher steel strengths or where economically justifiable: ASTM A242, A441, A514, A572, A588.

F. Structural steel and composite steel-concrete flexural members for aerial structures shall conform to the requirements of AASHTO LRFD.

G. The requirements governing LL deflections and structure deformations and settlements as outlined for reinforced and prestressed concrete design also apply to structural steel design.

H. Bolts: ASTM A325, unless otherwise shown on the contract drawings.

I. Refer to AISC Manual of Steel Construction, Load and Resistance and Factor Design, latest edition, Specification for Structural Joints Using ASTM A325 or A490 Bolts for use of bolts in snug-tightened, pretensioned, and slip critical joint applications.

J. Shop connections as detailed by the design-builder's or designer's lead structural engineer shall be welded unless otherwise directed by Metro. Weld in accordance with the current code or specifications of the AWS, as applicable.

K. A fracture critical guideway or bridge is a steel structure, subject to dynamic cyclic loading, which has at least one tension member or tension component of bending member (including those subject to reversal of stress), whose failure would be expected to result in the collapse of the bridge. It is the design-builder's or designer's lead structural engineer's responsibility to identify Fracture Critical Main Members, Secondary Members, and Components of Main Members in designing a new steel guideway or bridge and to designate or tabulate them explicitly on the contract documents (plans and/or Special Provisions). For further requirements see, Caltrans, *Memo to Designers*, Guidelines for Identification of Steel Bridge Members, latest edition.

5.4 UNDERGROUND GUIDEWAYS AND STRUCTURES

Underground guideways and structures are enclosed facilities, regardless of type or method of construction, that require special structural and geotechnical design considerations and may include lighting, ventilation, fire protection systems, and access and emergency egress capacity based on Metro's determination.

The structural shells for tunnels, consisting of plate elements, such as walls, base slab and roof, that form the earth resisting box along the longitudinal axis of these structures shall be contiguous moment resisting structural elements that give resistance to all static, dynamic, and seismic forces and distortions in accordance with these criteria through structural continuity, redundancy, and ductility for the service life specified. Consideration shall be given by Metro for underground guideways and structures implementing permanent ground support elements as final load resisting systems in conjunction with the completed permanent structures. All structural members carrying flexure or shear shall terminate in other structural members that transfer these forces through torsion, compression and tension such that no ductile hinges (limit design of structural members shall be allowed only for the MDE seismic load condition, see Metro Supplemental Seismic Design Criteria, Section 5 Appendix), are incorporated into the design of the main framing elements. Ductile hinges may be formed by the static forces between two shell structures where one connects at an opening of another.

Ductility is defined as the ratio between maximum displacement (rotation) and the start of yield displacement (rotation) for member system with structural and reinforcing steel.

For potential and computed zones of plastic hinge formation, the confining tie reinforcement for walls, pilasters and columns, shall not be less than provided by Caltrans BDS implemented AASHTO LRFD Section 5.10.11.4.1d and the compressive reinforcing shall not be less than the tensile reinforcing.

5.4.1 Tunnel Lining

This section covers the structural design, detailing, and construction of tunnel linings for tunnels focusing on mined or bored tunnels. Tunnel linings are structural systems installed after excavation to provide ground support, to maintain the tunnel opening, to limit the inflow of ground water and/or gas, to support appurtenances, and to provide a base for the final finished exposed surface of the tunnel. Tunnel linings can be used for initial stabilization of the excavation, permanent ground support or a combination of both. The materials used by Metro for tunnel linings are cast-in-place concrete, precast segmental concrete, fabricated steel and shotcrete.

Cast-in-place concrete linings are generally installed some time after an initial ground support. Cast-in-place concrete linings are used in hard rock tunnels and can be constructed of either reinforced or plain concrete.

Precast concrete linings are used as both initial and final ground support. Segments in the shape of circular arcs are precast and assembled inside the shield of a tunnel boring machine to form a ring. If necessary they can be used in a two-pass system as only the initial ground support. Initial support segments for a two-pass system may be lightly reinforced and rough cast. The second pass or final lining shall be cast-in-place concrete. Precast concrete linings may also be used in a one-pass system where the segments provide both the initial and final ground support. One pass precast segmental concrete linings shall be cast to strict tolerances and are provided with gaskets and may be bolted together to reduce the inflow of water and/or gas.

Fabricated steel linings are a type of segmental construction where steel plates are fabricated into arcs that typically are assembled inside the shield of a tunnel boring machine to form a ring. The fabricated steel lining may form the initial and final ground support. The segments are provided with gaskets to limit the inflow of ground water into the tunnel. Fabricated steel linings may be used to provide greater tunnel ductility in ground potentially subjected to large deformations from seismic activity.

Shotcrete may be used as an initial and/or final ground support for rock tunnels. With advances in shotcrete technology, permanent linings may be designed in conjunction with sequential excavation method (SEM) tunneling. Shotcrete may be applied over the exposed ground, reinforcing steel, welded wire fabric or lattice girders. It may be used in conjunction with rock bolts and dowels, it may contain steel or plastic fibers and it can be composed of a variety of mixes. It may be applied in layers until the design thickness is achieved.

Cross passages and refuge area are typically mined by hand after the main tunnel is excavated. The final lining for these areas, due to their unique shape and small areas, shall be lined with cast-in-place concrete unless otherwise/se allowed by Metro.

5.4.1.1 Load and Resistance Factor Design (LRFD)

The design of tunnel linings is not addressed in standard design codes. This section established the procedure for the design of tunnel linings utilizing the Federal Highway Administration (FHWA) FHWA-NHI-09-010, Chapter 10, Tunnel Lining, current edition which incorporates LRFD.

LRFD is a design philosophy that takes into account the variability in the prediction of loads and the variability in the behavior of structural elements. See also Section 5.2.17, LRFD Design Specifications, Design Life, and Limit States. This section is intended to assist the designer in the application of the LRFD specification to tunnel lining design and to provide a uniform interpretation of the FHWA document as it applies to tunnel linings.

5.4.2 Design Considerations

5.4.2.1 Lining Stiffness and Deformations

Tunnel linings are structural systems, but differ from other structural systems in that their interaction with the surrounding ground is an integral aspect of their behavior, stability, and overall load carrying capacity. The loss or lack of the support provided by the surrounding ground can lead to failure of the lining. The ability of the lining to deform under load is a function of the relative stiffness of the lining and surrounding ground. Frequently, a tunnel lining is more flexible than the surrounding ground. This flexibility allows the lining to deform as the surrounding ground deforms during and after the excavation of the tunnel. Likewise, this deformation mobilizes the strength and stability of the ground. The tunnel lining deformation allows the moments in the tunnel lining to redistribute such that the main load inside the lining is thrust or axial load. The most efficient tunnel lining is one that has high flexibility and ductility.

A tunnel lining maintains its stability and load carrying capacity through contact with the surrounding ground. As load is applied to one portion of the lining, the lining begins to deform, and in so doing, develops passive pressure along other portions of the lining. This passive pressure prevents the lining from buckling or collapsing. Ductility in the lining allows for the creation of *hinges* at points of high moment that relieve the moments in adjacent liner sections so that the primary load action becomes essentially axial force. This ductility is provided for in concrete by the formation of cracks in areas of flexural tension. Under reinforcing helps promote the formation of cracks. The joints in segmental concrete linings nevertheless must remain ductile during the cracking.

5.4.2.2 Durability

Tunnels can be exposed to extreme events such as fires resulting from incidents inside the tunnel. Tunnel lining design shall consider the effects of a fire on the lining. The lining should be able to withstand the heat of Metro specified fire intensity and period of time without loss of structural integrity. Protection from fire shall be determined by concrete cover on the reinforcing, additional tunnel finish and special treatment of the concrete mixes.

5.4.2.3 High Density Concrete

High density concrete shall be considered for tunnel applications. High density concrete is produced by using very finely ground cement and/or substituting various materials such as fly ash (see Section 5.3.6.1.B.6) or blast furnace slag for cement. It can limit the inflow of water and provide significant protection against chemical attach. High density concrete has low heat of conductivity which is beneficial in a fire.

5.4.2.4 Corrosion Protection

Corrosion protection aspects shall be evaluated during the design phase and shall be incorporated into the design. Corrosion is associated with steel products embedded in the concrete and otherwise used in tunnel applications. Ground water, ground chemicals, leaks, dissimilar metals, iron eating bacteria, and stray currents are all sources of corrosion in metals.

Corrosion protection shall take the form of increased cover for reinforcing, and concrete and metal coatings such as epoxies, powder coatings, paint or galvanizing. Insulation shall be installed between dissimilar metals and sources of stray currents. High density concrete can provide additional protection for reinforcing steel. See also Section 3.10 Corrosion Control.

5.4.2.5 Lining Joints

Cast-in-place linings shall have joints to provide relief from stresses induced by movements due to temperature changes. These linings shall have contraction joints every 30 feet and expansion joints every 120 feet. Expansion joints shall be provided where cut and cover portions of the tunnel transition to the tunneled or mined portion.

Segmental concrete linings do not require contraction joints and require expansion joints only at the cut and cover interface.

5.4.2.6 Specific Requirements for Flexible Earth-Tunnel Sections

A. General Requirements

1. These design criteria apply to flexible and semi-flexible precast concrete segmental and fabricated steel segmental tunnel liners.

2. Unless shown or specified otherwise, the liners may be bolted or unbolted on their longitudinal and circumferential joints.

3. In appropriate circumstances, the liners may be expanded against the ground. More generally, the annulus between the liner and the ground will be completely filled with cement grout placed immediately behind the tunnel boring machine.

4. Tapered liner rings shall be used to negotiate curves and correct vertical and horizontal alignment.

5. In tunneled sections below the water table, the liners must be capable of being made watertight or if necessary gastight, by means of sealing gaskets and/or caulking and bolt grommets.

6. No steel ring - timber spacer tunnel liners shall be used.

7. Threaded inserts shall be cast in all pre-cast and cast-in-place tunnel liners for equipment mounting.

B. Design of the Liners

1. The liners shall be designed to sustain all the loads to which they will be subjected with adequate factors of safety. Such loads will include:

   a. Handling loads as determined by the transport and handling system.

   b. Shield thrust ram loads as determined by the shield propulsion system.

c. Erection loads including external grouting loads.

d. Earth pressure, but in no case less than full overburden for depths of cover less than 50 feet, and no less than 6500 pounds per square foot for depths greater than 50 feet.

e. Hydrostatic pressure

f. Self-weight of the tunnel structure

g. Loads due to imperfect liner erection, but not less than 0.5% diametrical distortion.

h. Additional loads due to the driving of adjacent tunnels

i. Effects of tunnels breakouts at cross-passages, portals, and shafts

j. Live loads of vehicles moving in the tunnel or on the surface above it

k. Surcharge loads due to adjacent buildings

2. Seismic loads as indicated in the "Metro Supplemental Criteria for Seismic Design of Underground Structures, Appendix, Chapter 3, Part B."

3. Provisions shall be made in the liner segments for corrosion prevention and the elimination of stray currents from the surrounding ground area.

4. Provisions for ground structure interaction and lateral support of surrounding ground shall be included.

5.4.2.7 Specific Requirements for Rock Tunnel Liners

A. General Requirements

1. These design criteria apply to cast-in-place concrete liners and flexible or semi-flexible precast concrete segmental liners erected directly behind the tunneling machine.

2. For the cast-in-place concrete liners, temporary support may be required during the excavation phase of the tunneling process. This temporary support, in general, will be provided by steel arch ribs at centers to suit rock conditions. When and if rock conditions permit, these may be replaced by lattice girders, resin-anchored rock bolts at centers to suit rock conditions, shotcrete applied to the rock surface, or combinations of the above.

3. Unless shown, specified, or otherwise directed, the precast concrete segmental liners may be bolted or unbolted on their longitudinal and circumferential joints.

4. In appropriate circumstances the segmental liners may be expanded against the ground. More generally, the annulus between the liner and the ground will be completely filled with cement grout placed immediately behind the tunnel boring machine.

5. Tapered segmental liner rings shall be used to negotiate curves and correct vertical and horizontal alignment.

6. In tunneled sections below the water table, the liners must be capable of being made watertight by means of sealing gaskets, duct sealants, caulking or rock grouting or designed to incorporate a drainage system to relieve hydrostatic pressures behind the liner to drain to an invert drain in the tunnels.

7. No steel ring - timber spacer tunnel liners shall be used.

8. Threaded inserts shall be cast in all pre-cast and cast-in-place tunnel liners for equipment mounting.

B. Design of the Liners

1. The temporary support systems shall be designed to sustain all the loads to which they will be subjected with adequate factors of safety for temporary conditions. Such loads will include:

a. Rock load determined by rock condition

b. Self-weight

c. Additional loads due to the driving of adjacent tunnels.

d. Grouting pressures

2. The cast-in-place liners shall be designed to sustain all the loads to which they will be subjected with adequate factors of safety without beneficial effects from the initial support system. Such loads will include:

a. Rock loads based on considerations of rock condition

b. Hydrostatic pressure either total or residual

c. Additional loads due to the driving of adjacent tunnels (if applicable)

d. Live loads of vehicles moving in the tunnel or on the surface above it

e. Seismic loads as indicated in the "Supplemental Criteria for Seismic Design of Underground Structures."

3. The precast segmental liners shall be designed to sustain all the loads to which they will be subjected with adequate factors of safety as defined by these criteria. Such loads will include:

a. Handling loads as determined by the transport and handling system.

b. Shield thrust ram loads if applicable as determined by the shield propulsion system.

c. Erection loads including external grouting loads

d. Rock loads based on considerations of rock condition

e. Hydrostatic pressure either total or residual

f. Self-weight of the tunnel structure

g. Loads due to imperfect liner erection

h. Additional loads due to the driving of adjacent tunnels

i. Live loads of vehicles moving in the tunnel

j. Seismic loads as indicated in Metro Seismic Design Criteria, Section 5B

5.4.3 Detailed Structural Design of Liners

The structural design of liners shall be governed by the Caltrans implemented AASHTO LRFD Bridge Design Specifications, Latest Edition, with California Amendments The above includes the latest edition of AASHTO Guide Specifications for LRFD Seismic Bridge Design. All the above is referred to throughout these criteria as "Caltrans BDS." For a summary of applicable codes, see Section 5.1.3, Metro Supplemental Seismic Design Criteria.

This section also provides the design procedure based on LRFD specification for structural plain concrete.

5.4.3.1 Loads

The loads to be considered in the design of structures are given in Section 5.2, Loads and Conditions. In addition to the loads discussed there, the following loads have additional special significance in the design of tunnels:

ES = Earth surcharge load: This is the vertical earth load due to fill over the structure that was placed above the original ground line. A minimum surcharge load of 400 psf shall be used in the design of tunnels. If there is a potential for future development adjacent to the tunnel structure, the surcharge from the actual development shall be used in the design of the structure. In lieu of a well defined loading, a minimum value of 1000psf shall be used when future development is a possibility.

LS = Live Load Surcharge: This load is to be applied to the lining of tunnels that are construction under other roadways, rail lines, runways or other facilities that carry moving vehicles. This is a uniformly distributed load that simulated the distribution of wheel loads through the earth fill. This load shall also be considered near the interface between the cut and cover approaches and the mined tunnel section.

DD = Downdrag: This load comprises the vertical force applied to the exterior of the lining that can result from the subsidence of the surrounding soil due to the subsidence of the in-situ soil below the bottom of the tunnel. This load will occur only when backfill in excess to the original ground elevation is placed over the tunnel or a structure is constructed over the tunnel.

WA = Buoyancy: For a tunnel, the overall weight of the structure is usually less than the soil it is replacing. Nevertheless, this is the reverse of downdrag and shall be investigated in cases of low ground cover and high water table.

5.4.3.2 Load Factors and Loading Combinations

The tunnel linings shall be designed for the appropriate load combination limit states and load factors as specified under Section 5.2.20, Loading Factors and Loading Combination, provided by this criteria. Additionally, for precast segmental liners, consider load combination in AASHTO LRFD equation 3.4.1-2 for service limit state (Service VI in Table 5-2).

The load case for the design of linings for mined tunnels given in Table 5-3 shall be used:

Table 5-3 Loading Combinations and Load Factors for Tunnels

Load Combination Limit State	Permanent Loads			Transient Loads		Loads Due to Volumetric Change		Exceptional Loads Use One of These at a Time	
	DC DD DW EH EV ES EL CR SH	SE	WA	LL LL_{HRV} LL_{LRV} IMV IMH CE LF/BR LS	LL_{HP} LL_{LP} IM CE	FR	TU**	TG	EQ
Strength I	γ_p	γ_{SE}	γ_p	1.75	—	1.00	0.50/1.20	—	—
Strength II	γ_p	γ_{SE}	γ_p	—	1.35	1.00	0.50/1.20	—	—
Extreme Event I	1.00	—	1.00	1.0*	—	1.00	—	—	1.00
Extreme Event IA	1.00	—	1.00	1.0*	—	1.00	—	—	1.00
Service I	1.00	γ_{SE}	1.00	1.00	—	1.00	1.00/1.20	γ_{TG}	—

* Live load from Heavy Rail Vehicle (HRV) and Light Rail Vehicle (LRV) to be loaded on one track only.

** Larger value shall be used for deformations and smaller value for all other effects.

γ_ρ Values, See Caltrans BDS referenced AASHTO Table 3.4.1-2, and Table 3.4.1-3 Load Factors for Permanent Loads, except as noted herein.

γ_ρ Values for CR and SH; See Caltrans BDS referenced AASHTO Table 3.4.1-3, Load Factors for Permanent Loads Due to Superimposed Deformations.

Y_ρ Values for WA shall equal the value for DC.

γ_{TG} Values for Service I shall be 0.5.

γ_{SE} The load factors for settlement should be considered on a project-specific basis in accordance with the GPR.

5.4.3.3 Design Criteria for Plain Concrete Members

Caltrans BDS does not address plain concrete. The following design procedure shall be followed for structural plain concrete tunnel linings.

Calculate the moment capacity on the compression face of the lining as follows:

$$\Phi M_{nC} = \Phi 0.85 f_c' S$$

Where:

M_{nC} = The nominal resistance of the compression face of the concrete

Φ = 0.55 for plain concrete

f_c' = 28 day compressive strength of concrete

S = The section modulus of the lining based on the gross uncracked section

Calculate the moment capacity on the tension face of the lining as follows:

$$\Phi M_{nT} = 5\Phi(f_c')^{1/2}S$$

Where:

M_{nT} = The nominal resistance of the compression face of the concrete

Φ = 0.55 for plain concrete

f_c' = 28 day compressive strength of concrete

S = The section modulus of the lining based on the gross uncracked section

Calculate the compressive strength of the lining as follows:

$$\Phi P_C = \Phi 0.6\ f_c' A$$

Where:

P_C = The nominal resistance of the lining in compression

Φ = 0.55 for plain concrete

f_c' = 28 day compressive strength of concrete

A = The cross sectional area of the lining section

Check the compression face as follows:

$$Q_A/\Phi P_C + Q_M/\Phi M_{nC} \leq 1$$

Where:

Q_A = the axial load force effect modified by the appropriate factors

Q_M = the moment force effect modified by the appropriate factors

Calculate the tension strength of the lining as follows:

$$\Phi P_T = 5\Phi(f_c')^{1/2}$$

Where:

P_T = The nominal resistance of the lining in tension

Φ = 0.55 for plain concrete

f_c' = 28 day compressive strength of concrete

Check the tension face as follows:

$$Q_M/S - Q_A/\Phi A \leq \Phi P_T$$

Where the values of the variables are those described above.

The shear strength of the lining is calculated as follows:

$$\Phi V_n = \Phi 1.33(f_c')^{1/2} b_w h$$

Where:

V_n = The nominal resistance of the lining in compression

Φ = 0.55 for plain concrete

f_c' = 28 day compressive strength of concrete

b_w = The length of the tunnel lining under design

h = the design thickness of the tunnel lining

This design method is adapted from LRFD from the provision for structural plain concrete from ACI 318.

5.4.3.4 Structural Analysis

Any method of structural analysis used by the designer shall be approved by Metro. Some widely accepted structural analysis methods are described in this section.

Beam Spring Models – A general purpose structural analysis program can be used to model the soil structure interaction. This method is known as the beam spring model. The computer model is constructed by placing a joint or node at points along the centroid of the lining. These nodes are joined by straight beam members that approximate the lining shape by a series of chords. When constructing the model, the chord lengths should be about the same as the lining thickness. A subtended angle dimension of about 60/R, where R is the radius of the tunnel in feet, should produce acceptable results. Since the compressive forces are generally large enough to have compression over the entire thickness of the lining, the area and moment of inertia may usually be calculated using the gross, uncracked dimension of the lining.

The surrounding ground is modeled by placing a spring support at each joint. Springs can be placed in radial and tangential directions. The numerical value of the spring constant at each support is calculated from the modulus of subgrade reaction of the surrounding ground multiplied by the tributary length of the lining on each side of the spring. Parametric studies that vary the ground conditions and the spring constants should be preformed to determine the worst case scenario for the lining.

Empirical Method for Soft Ground – For the beam spring model method, trail and error computations that adjust the distortion of the ring in order to obtain a final solution in which the ring and the ground distortions are compatible are required for realistic soil structure interaction results.

Using this method, the thrust in the tunnel lining is calculated by the formula:

$$T = wR$$

Where:

T = the thrust in the tunnel lining

w = the earth pressure at the spring line of the tunnel due to all sources

R = the radius of the tunnel

The percentage of radius change to be used is a function of the type of soil. Values for this percentage estimated by Birger Schmidt are shown in Table 5-4.

Table 5-4 Percentage of Lining Radius Change in Soil

Soil Type	ΔR/R – Range
Stiff to Hard Clays	0.15 – 0.40%
Soft Clays or Silts	0.25 – 0.75%
Dense or Cohesive Soils, Most Residual Soils	0.05 – 0.25%
Loose Sands	0.10 – 0.35%

Notes:

1. Add 0.1 to 0.3 percent for tunnels in compressed air, depending on air pressure.
2. Add appropriate distortion for effects such as passing neighbor tunnel.
3. Values assume reasonable care in construction, and standard excavation and lining methods.

The resulting bending moment in the lining is calculated using the following formula:

$$M = 3EI/R \times \Delta R/R$$

Where:

M = the calculated bending moment

R = the radius of the centroid of the lining

ΔR = tunnel radius change

E = the modulus of elasticity of the lining material

I = the effective moment of inertia of the lining section

The effective moment of inertia can be calculated for precast segmental linings using the following formula developed by Muir Wood:

$$I_e = I_j + I(4/n)^2$$

Where:

Ie = the effective moment of inertia

Ij = the joint moment of inertia

I = the moment of inertia of the gross lining section

n = the number of joints in the lining ring

The moment of inertia for the uncracked section should be used for cast-in-place concrete linings, but should be changed to the cracked section if the first run shows tension in the segment concrete.

Numerical Methods – Commercial software is available to model both the lining and the surrounding ground as a continuum utilizing a two or three dimensional finite element or finite difference approach. These programs provide structural beam and shell elements to be used to model the tunnel lining.

5.4.4 Ventilations Shafts

The permanent shaft walls shall be reinforced concrete. Loadings imposed on the shaft by the surrounding ground shall be as given for underground structures and consistent with the shaft configuration. Shafts shall be inclined less than 45 degrees from the vertical.

5.4.5 Tunnel Break-Outs

Permanent walls for tunnel break-outs in shafts, cross-passages, or any other location shall be in reinforced concrete. For tunnels lined with pre-cast segmental tunnel liners, requirements of specially segmented rings to suit break-out configurations shall be determined by the Project Geotechnical Engineer. Cross-passages may be combined with other on-line structures such as pump and ventilation structures. Refer to NFPA 130, Fixed Guideway Transit Systems.

5.4.6 Portals and U-Sections

Tunnels and box section entrance portals shall be designed in a manner to minimize the rate-of-change of pressure on a train passing through the portal.

The pressure rise is a function of both the cross-sectional area of the portal entrance and the entrance speed of the train (see Figure 5-6).

5.4.6.1 Acceptable Design Methods

A. Provide the entrance with a flared transition so that the increase in cross-sectional area approximates the cross-section of a six degree conical flare starting at the constant area section of the tunnel or box and extending to the portal opening. This flared transition can be formed using any combination of tapers on the top and sides, provided no plane or surface of the transition section is at an angle in excess of six degrees relative to the center line of the tunnel and provided the side tapers are symmetrical with the center line. For the required length of the flared transition, see Figure 5-7 and for the required cross-sectional area at the portal, see Figure 5-8

B. Design both the top and vertical sides of the entrance without a flare and provide a tapering slot in the top. From a one-foot minimum width at the constant area section the slot should increase to a maximum at the portal at a taper rate of 12 feet per 100 feet of length. The slot opening should, therefore, be 13 feet wide at the portal for 100 foot long transition, or seven feet wide at portal for 50 foot long transition. For the required length of transition, see Figure 5-7.

5.4.6.2 Exceptions

Exceptions that do not require special transition portal are:

A. Tunnels of a length less than 200 feet

B. Single track horseshoe tunnels with design train speed of 45 mph or lower

C. Box sections and single track circular tunnels with design train speed of 40 mph or lower

D. Portals at underground stations.

5.4.6.3 General Requirements

A. In locating portals and determining the ends of U-sections and walls, consideration shall be given to providing protection against flooding resulting from local storm runoff.

B. Adequate provision shall be made for resistance to hydrostatic uplift. Adequate provision shall be made for immediate and effective removal of water from rainfall, drainage, groundwater seepage, or any other source.

C. U-sections, with both walls continuous with a full-width base slab, shall be used for open-cut sections where the top of rail is less than 4' above the maximum groundwater table. Above that level, independent reinforced concrete cantilever retaining walls may be considered for design in accordance with the provisions of Article 5.4.8.

D. U-sections may be ~~analyzed~~ **analysed** as continuous structures on elastic foundations. If at any station the two walls are of unequal heights, then the factor of safety against sliding shall be a minimum of:

1. 1.50 with no passive resistance of the soil.

2. 2.00 with passive resistance of the soil.

E. Wall thickness for U-sections shall be designed by using:

1. The geotechnical soils report recommendations for coefficient of lateral earth pressure, at-rest case.

2. Hydrostatic pressure.

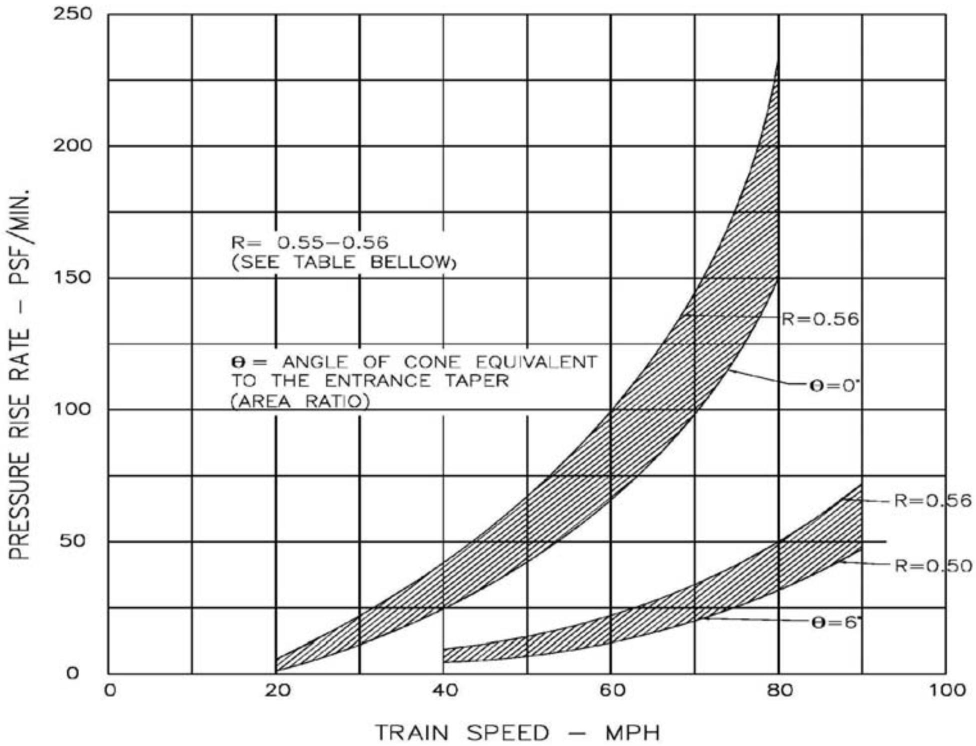
3. Surcharge effects.

F. U-section grade slab design thickness shall be 6" greater than the wall thickness, with a minimum thickness of 24". If the weight of the grade slab (in psf) is less than 40% of the hydrostatic head (in psf) as measured from the bottom of the grade slab, then the grade slab shall be designed for uplift pressure.

G. If, at the last U-section segment away from the portals, the abutting at-grade trackway does not consist of a track slab, then a depressed approach slab shall be provided to permit the construction of tie-and-ballast trackbed up to the end of the U-section base slab that avoids a sharp break in support at that point.

H. Seismic loads as given in Metro Supplemental Design Criteria, Chapter 3 Part A and B.

Figure 5-6
PRESSURE RISE RATE VS. TRAIN SPEED



ESTIMATED PRESSURE RISE RATE VS. TRAIN SPEED FOR TUNNEL ENTRANCES WITH NO TAPER (Θ=0°) AND WITH 5° CONICAL TAPER (Θ=6°), OR EQUIVALENT.

TYPE OF TUNNEL	CROSS–SECTIONAL AREA	R
SINGLE BOX, CUT AND COVER	174 sq. fT.	.55
DOUBLEE BOX, CUT AND COVER	171	.56
CIRCULAR EARTH TUNNEL	203	.47
HORSESHOE ROCK TUNNEL	180	.53

$$R = \frac{\text{CAR CROSS – SECTIONAL AREA}}{\text{TUNNEL CROSS – SECTIONAL AREA}}$$

Figure 5-7 TUNNEL TRANSITION LENGTH VS TRAIN SPEED

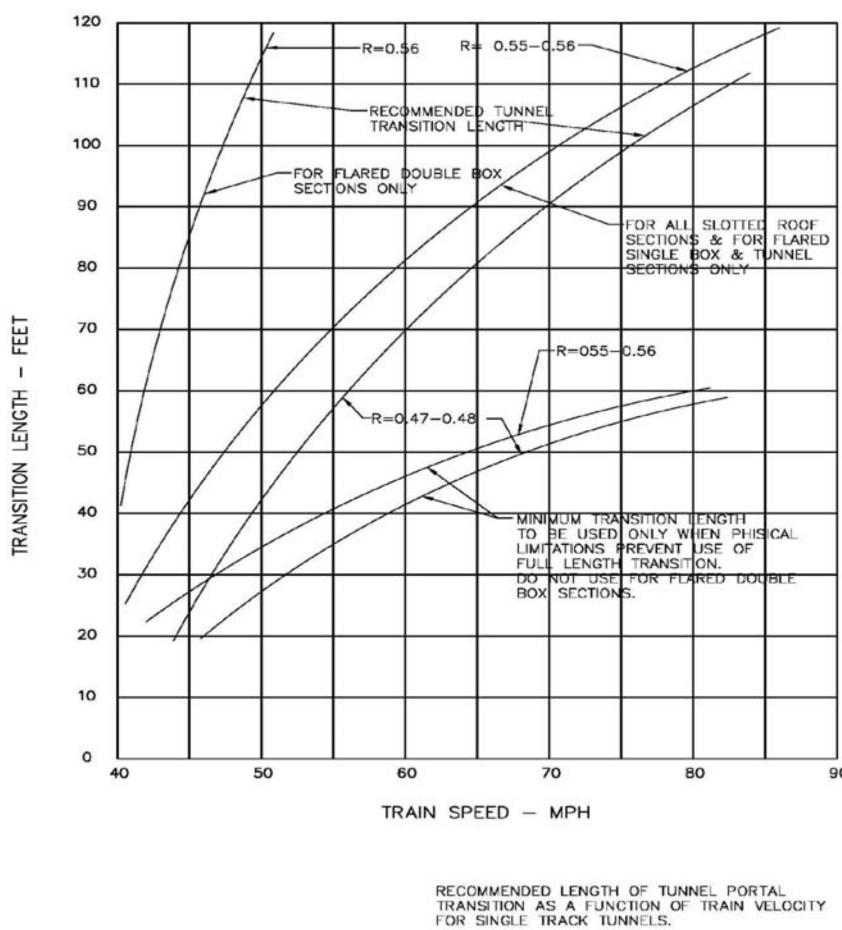
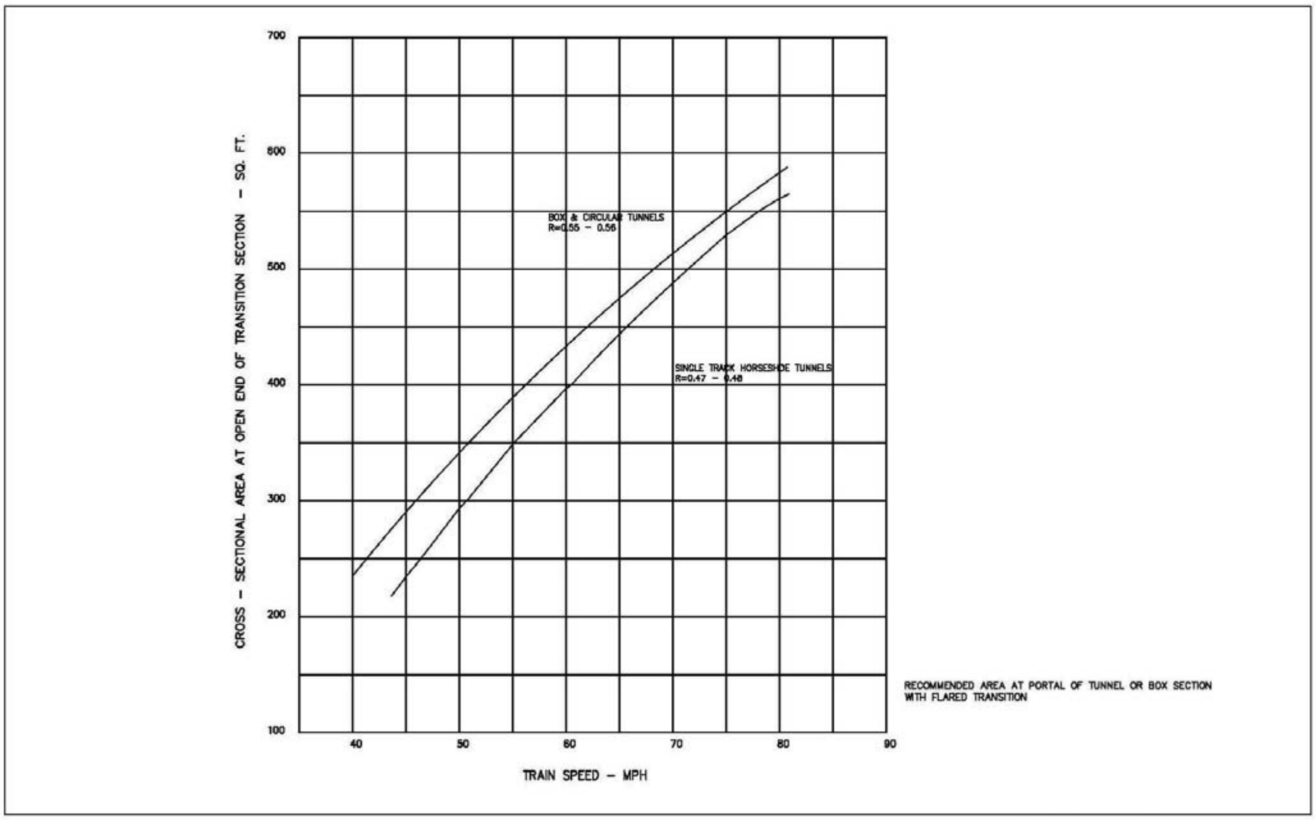


Figure 5-8
RECOMMENDED AREA AT PORTAL OF TUNNEL OR BOX SECTION WITH FLARED TRANSITION



5.4.7 Reinforced Concrete Box Station Sections

Except as otherwise provided by Sections 5.1.2, 5.1.3.C.4, and by this section, underground station structures and their appurtenant structural elements such as entrances shall be designed in accordance with Caltrans BDS referenced AASHTO specifications as outlined in Sections 5.1.2 and 5.1.3.

Subsurface exploration shall be carried out to determine the presence and influence of geologic and environmental conditions that may affect the performance of station structures and reported by one or more geotechnical data reports (GDR). See Section 5.6.2.3.

A. Load combinations and load factors to be used are those provided by Table 5-5. Load resistant factors to be used are those provided by Caltrans BDS and their referenced AASHTO Tables 3.4.1-2, 3.4.1-3, and 12.5.5-1. In addition, the effects of EH, EV, ES, LS, DD, DW, and WA shall be applied simultaneously in all their maximum and minimum values to produce the envelope of moment, torsion, shear, and axial force to produce the greatest demands to the structural framing. These load values shall cover the forces on the station structure at all phases of construction. See Caltrans BDS referenced AASHTO Section 5.14.2.3.

   Final ground induced pressures and design assumptions for soil-structure interaction shall be provided by the GBR, Metro SSDC, Section 1.5. Conceptual and preliminary assumptions are addressed by Section 5.6, Geotechnical.

B. Foundation Pressures

   Vertical pressure on foundation slabs may be divided into hydrostatic and earth pressure components. The hydrostatic component shall be distributed across the width of the foundation in proportion to the depth of each portion of the basic slab below the design groundwater table.

   Distribution of the earth pressure moment shall be based on specified construction procedures, and will include elastic and plastic subgrade reaction foundation effects.

C. For design, the horizontal earth pressure distribution diagram for multiple braced flexible walls shall be the trapezoidal pressure diagram as given on the Contract Drawings. Compression forces shall not be considered in shear design of the top and bottom slab in box sections.

D. In evaluating the design for temporary loadings produced by construction conditions such as the removal of horizontal struts, consideration shall be given to:

   1. Allowable increase in stresses due to the temporary nature of the loading.

   2. Creep in the concrete.

   3. Effect of soil arching.

4. Wall and slab flexibility.

E. Where restrutting is to be used, design calculations submitted shall be submitted for approval and must reflect proper consideration of such aspects as magnitude of preload in replacement struts, crushing of packing, and thermal-induced stress and deflection of the permanent structure. The design shall also detail the proposed instrumentation and monitoring thereof so as to ensure that the permanent structure will not be overstressed or otherwise damaged.

F. In all cases, the design for support of excavation must reflect any limitations inherent in the design of the permanent structure.

G. Adequate provisions shall be made for corrosion control in accordance with specifications and in consultation with the corrosion consultant.

Table 5-5 Loading Combinations and Load Factors for Underground Station Structures

Load Combination Limit State	Permanent Loads			Transient Loads		Loads Due to Volumetric Change		Exceptional Loads Use One of These at a Time	
	DC DD DW EH EV ES EL PS CR SH	SE	WA	LL LL_{HL93} LL_{HRV} LL_{LRV} IMV IMH CE LF/BR PL LS	LL_{PERMIT} LL_{HP} LL_{LP} IM CE	FR	TU** TTR** TLR**	TG	EQ
Strength I	γ_p	γ_{SE}	γ_p	1.75	—	1.00	0.50/1.20	—	—
Strength II	γ_p	γ_{SE}	γ_p	—	1.35	1.00	0.50/1.20	—	—
Extreme Event I	1.00	—	1.00	1.0*	—	1.00	—	—	1.00
Extreme Event IA	1.00	—	1.00	1.0*	—	1.00	—	—	1.00
Service I	1.00	γ_{SE}	1.00	1.00	—	1.00	1.00/1.20	γ_{TG}	—
Service II	1.00	—	1.00	1.30	—	1.00	1.00/1.20	—	—
Service III	1.00	γ_{SE}	1.00	0.8	—	1.00	1.00/1.20	γ_{TG}	—
Fatigue I **LL_{HL93}, LL_{HRV}, LL_{LRV}, IMV, IMH, & CE Only**	—	—	—	0.75	0.75	—	—	—	—
Fatigue II **LL_{PERMIT}, LL_{HRV}, LL_{LRV}, & IM**	—	—	—	—	1.00	—	—	—	—

* Live load from Heavy Rail Vehicle (HRV) and Light Rail Vehicle (LRV) to be loaded on one track only.

** Larger value shall be used for deformations and smaller value for all other effects.

γ_p Values, See Caltrans BDS referenced AASHTO Table 3.4.1-2 and Table 3.4.1-3, Load Factors for Permanent Loads, except as noted herein.

Y_p Values for WA shall equal the value for DC.

γ_p Values for PS, CR, and SH; See Caltrans BDS referenced AASHTO Table 3.4.1-3, Load Factors for Permanent Loads Due to Superimposed Deformations.
γ_p Values for EL shall equal the value for DC.
γ_{TG} Values for Service I, & III shall be 0.5,.
γ_{SE} The load factors for settlement should be considered on a project-specific basis in accordance with the GPR.

5.4.8 Reinforced Concrete Retaining Walls

Retaining walls above 20 ft-0" in height shall be designed on the basis of specific soils information relating to the backfill material using an acceptable method provided in Section 5.6 Geotechnical.

5.4.9 Shafts

Permanent shaft walls shall be reinforced concrete. Loads imposed on the shaft by the surrounding medium and applicable surface loadings shall be determined by using an acceptable method provided in Section 5.6 Geotechnical.

5.4.10 Miscellaneous Structures

5.4.10.1 Gratings

The following grating types shall be adopted as standards for use in Metro Projects:

A.	For light loading	For general use not subject to vehicular loads
	Bearing bars	1-1/4 in. x 3/16 in. on 1-3/16 in. centers
	Crossbars	4 in. centers
	Maximum allowable deflection	1/200 span
	Grating type	Rectangular plain
	Grating type	Rectangular-plain
	Material	Steel, hot dip galvanized
B.	For Sidewalks	
	Bearing Bars	2 ½" x 3/16" on 15/32" Centers
	Crossbars	4" Centers
	Design loading	900 psf
	Maximum allowable deflection	1/300 span
	Grating type	Rectangular-plain

Material	Steel, hot dip galvanized, with non-slip granular finish on walking surface

C. For heavy loading — Grating subject to vehicle wheel loads

Bearing bars	4 in. x 1/4 in. on 1-3/16 in. centers
Crossbars	4 in. centers
Design loading	AASHTO HL93
Maximum allowable deflection	1/300 span
Grating type	Rectangular-plain
Material	Steel, hot dip galvanized

5.4.10.2 Emergency Access Shafts

A. Access shall be provided to the subway as specified in Fire/Life Safety Criteria.

B. Hatches on access shafts shall be readily unlatched from the inside of the subway by means of panic hardware and opened by means of a key-operated device from outside the subway in accordance with the Fire/Life Safety Criteria. Continuous handrails shall be provided in access shaft passageways as well as on stairways.

C. Access hatches shall be protected from surface water. Allowances must be made to divert surface water to the drainage system, away from the hatches.

D. Where doors are required, they shall open in the exit direction at the subway level and at the surface level. Where locks are required, they shall be provided with panic hardware. Doors shall also meet the fire rating specified in the local codes.

E. All doors and hatches shall be provided with the means for future installation of intrusion detection systems.

5.4.10.3 Parapets

Where parapets are used, they shall be designed to withstand dead load, wind load, force due to thermal expansion and contraction, shrinkage force, and earthquake forces equal to the full dead load of the parapet acting at the center of mass of the component parts.

5.4.10.4 Air Pressure Due to Moving Trains

In underground structures air pressure is produced by running trains. Components including walls, ceilings, doors and ductwork shall be designed to meet or exceed these pressures, shown in Section 6 Architectural, Figure 6.1.

5.4.10.5 Elevators

The surface structure shall be designed for the loads described below:

A. Dead load of structure.

B. Live load of 100 plf applied at the free edges of the frame.

C. Wind load of 40 psf on windward side.

D. For traction type elevators, the surface structure shall be designed to support elevator beams. The end reaction of the elevator beams shall be 18,000 lb minimum. The location of the elevator beams varies with the type of elevator and its relative machine room location. The designer shall coordinate with elevator manufacturers regarding elevator beam locations.

5.4.10.6 Escalators

The support elements shall be designed for the end reactions from the escalators. The end reactions will be provided to the designer by Metro.

5.4.11 Adjacent Public Buildings

A. New building by private developers representing commercial interests or other public agencies that are planning pedestrian entrance access to Metro facilities must have their designs reviewed by Metro. It is the general policy of Metro to review designs on a case-by-case basis. This includes not only plans for physical attachment, but also all new construction within the influence zone of the existing Metro facilities.

B. A lateral separation of 8 feet shall be maintained between the finished exterior walls and foundations of underground Metro station facilities and those of new building construction if seismic isolation is to be assumed. Similarly, a vertical separation of 8 feet should be maintained between the finished exterior roofs of underground Metro station facilities and the mat or spread footing foundations of new building construction. All new construction closer than this, the developers shall demonstrate seismic isolation by submitting calculations.

C. High-rise tubular buildings, that create large perimeter loads on their foundations during seismic events, shall be ~~analyzed~~ **analysed** as special cases. This analysis shall demonstrate the pressures created by the building foundations do not exceed the design loads on Metro facilities using accepted geotechnical techniques of analysis.

D. Where joint development passageways interface directly with Metro station facilities, calculations shall be provided to demonstrate the new building elements have sufficient seismic and differential settlement ductility, as not to cause overstress to existing or new structural elements. The details shall be provided to demonstrate that waterproof integrity of the new construction with the existing station structure can be maintained for its anticipated service life.

In areas where the geotechnical investigation for Metro facilities, or that of the planned or joint development construction suggests a potential for the seismic consolidation or liquefaction of the soil, a special structural and geotechnical analysis of both designs shall be undertaken by the developers.

E. Temporary support of Metro facilities during the adjacent excavation for new buildings will be such that at any level, the station shell static lateral displacement shall not exceed 0.0005 times its overall height above the bottom of the base slab. The lateral forces used for the design of temporary excavation support shall consider both the static and dynamic loads for which Metro facility was designed. Where temporary support elements are used, details shall be provided to demonstrate that membrane waterproofing shall not be damaged.

F. Subsurface areas of new buildings where the public has access or that cannot be guaranteed as a secure area, such as parking garages and commercial storage and warehousing, will be treated as areas of potential explosion. NFPA 130, Standard for Fixed Guideway Transit Systems, life safety separation criteria will be applied that assumes such spaces contain Class I flammable, or Class II or Class III combustible liquids. For structural and other considerations, separation and isolation for blast shall be treated the same as for seismic, and the more restrictive shall be applied.

G. All static, dynamic, and geotechnical zone of influence calculations provided in support of the above criteria, will be sealed by a structural or Geotechnical engineer licensed in the State of California. Zone of influence calculation shall be based on intergranular soil pressures that consider the worst possible hydrostatic condition that might occur during the service life of Metro facilities.

5.4.12 Reinforced and Prestressed Concrete

5.4.12.1 Minimum Concrete Design Strengths

A. For all underground reinforced concrete cast-in-place structures including cut and cover box lines and stations, abutments, retaining walls, shafts, cross-passageways, portals, U- sections, spread footings, piles, drilled-in caissons, and basement walls:

- f'c = 4,000 psi.

B. For all aboveground reinforced concrete cast-in-place structures:

- f'c = 4,000 psi.

C. For prestressed concrete

- f'c = 6,000 psi.

D. For precast prestressed members

- f'c = 5,000 psi.

E. For all building foundations, floor slabs, pits and other miscellaneous foundations at yards and shops, miscellaneous foundations other than those specified, and station platform foundations

- f'c = 3,000 psi.

F. In certain cases, strengths of concrete other than those specified above might be required. These cases will be as recommended by the Contractor’s Designer and accepted by Metro.

5.4.12.2 Reinforcing and Prestressing Steel

A. Comply with ASTM A706 for reinforcement resisting earthquake-induced flexural and axial forces in frame members and in wall boundary members. ASTM A615 grades 40 and 60 reinforcement are allowed in these members if (a) the actual yield strength based on mill tests does not exceed the specified yield strength by more than 18,000 psi (retests shall not exceed this value by more than an additional 3000 psi) and (b) the ratio of the actual ultimate tensile stress to the actual tensile yield strength is not less than 1.25. For aerial structures and bridges conform to the requirements of Caltrans BDS.

B. Prestressing steel: See Section 5.3.6.1

C. To the extent possible, space main reinforcing bars at 6 in., 9 in., or 12 in. on centers in elements of retaining walls, bridge structures, and stations. Exceptions to this rule include columns, stairways, and thin slabs.

D. Standardization of spacing is intended to simplify design details, checking of bar placement, and field inspection. Spacing should also consider ease of concrete placement, room for embedded items, decrease in concrete coverage due to lapped splices, and the blockages that might occur by crossings of closely spaced reinforcement. Bar sizes should be selected to avoid crowding, particularly where larger size rebar is used. When bars are lap spliced, the spacing of the rebar must be such that a clear space of at least three inches is available between the lapped pairs to permit entry of vibrator.

E. The determination of soil/water corrosive conditions shall follow Caltrans Bridge Design Specifications Chapter 8.22. The various requirements contained therein will also apply, such as: admixtures additions, use of epoxied reinforcement, minimum concrete cover requirements, etc. See also Section 3.10 Corrosion Control.

5.4.12.3 Methods of Design

A. Underground structures and parts thereof shall be designed in accordance with strength design, load and resistance factor design, allowable stress design, or empirical design, as permitted by the applicable material chapters of the codes listed under Section 5.1, and as provided by Section 5.6 Geotechnical.

B. Loads and forces not covered in the above codes shall be subject to the approval of Metro.

C. Elastic and plastic subgrade reaction shall be considered for both vertical and horizontal loads during construction and for the completed structure.

5.4.12.4 Joints in Cut-and-Cover Structures

A. To promote water tightness and structural integrity and in view of relatively uniform internal temperatures in the massive main members of subway structures, expansion or contraction joints are not to be provided within the subway line or station structures. Construction joints are required as follows:

At locations of major change in structure section, e.g., from cut-and-cover line to station or from cut-and-cover structure to open-cut structure, construction joints shall be provided.

Where a cut and cover box line section meets a station section: Design the connection either to absorb any differential movements or to transmit the forces that may occur under any design condition. In all cases, give thorough consideration to water-tightness.

B. To control shrinkage stresses in monolithically poured concrete slabs and walls and to minimize cracking, provide construction joints at a spacing not exceeding 50 ft, and closer if appropriate to the framing construction. Design joints to have continuous reinforcing steel, keys, or other positive means of shear transfer. Use nonmetallic water stops for all exterior elements in contact with soil or rock.

C. Do not use expansion or contraction joints in cut-and-cover structures. Provide continuous temperature and shrinkage reinforcement, as required by applicable specifications and codes, in all walls and slabs of these underground structures.

5.4.12.5 Water and Gas Proofing

A. Stations

Provide external membrane water and gas proofing using High Density Polyethylene (HDPE also referred to as HCR, Hydrocarbon Resistant Membrane) entirely around cut-and-cover station structures. Show boundary condition details, such as reglets, flashing, and laps, on the construction drawings. Where cut and cover structures are to meet existing underground stations or tunnels, ensure waterproofing is continuous, and no leakage is detectable.

B. Equipment Rooms

Where train control rooms, electrical rooms, and auxiliary equipment spaces have base slabs, roofs and walls in contact with earth, waterproof using a HDPE external membrane. Where the base floor is subjected to hydrostatic pressure, slope the floor to drain and install equipment on raised pads. Locate equipment to permit repair of leaks while the equipment is in operation.

Use external membrane waterproofing for substations, switchgear, ventilation and sump pump rooms, and similar rooms as described above.

Place conduits leading from walls or roofs of any of the above spaces so as to prevent water running in or along the conduit to the equipment.

C. Tunnels

Tunnel linings are structural systems designed to limit the inflow of water by the density of the concrete, the use of gaskets at the joints, and by careful installation of the liner elements. For tunnels with initial liners and those in rock, water and gas proofing is achieved by external membrane water and gas proof linings.

D. Repairs

Make provision for injection sealing of leaks at the tunnel/station interface.

Give special consideration to design details and construction sequences for reduction of cracking. Specify high density concrete to promote impermeability through control of the water cement ratio, the proportions of cement and Pozzolan materials, and the placement and curing temperature of the concrete.

5.4.12.6 Architectural Considerations

To ensure uniformity of structural concrete color in public areas of the stations, Standardize concrete mix and strength, the aggregate source, and the brand of cement to be used in any given area. This applies to all concrete exposed to public view within the stations or to the concrete exposed to view from outside the stations.

5.4.13 Structural Steel

Use the following steel:

5.4.13.1 Structural Steel

For normal use - ASTM A36, or ASTM A50.

5.4.13.2 High-Strength Structural Steel

For uses requiring higher-strength steels or where economically justifiable - ASTM A242, A441, A514, A572, A588.

5.4.13.3 Connections

A. Shop connections as detailed by the Contractor's designer shall be welded unless otherwise approved by Metro. Weld in accordance with the current code or specifications of the American Welding Society, Inc., D1.1 Series, as applicable.

B. Design field connections for high-strength bolts or welding. Use high-strength ASTM A325 bolts.

5.5 SURFACE FACILITIES

In the County and City of Los Angeles, apply The Los Angeles County Building Code, as applicable. For structures other than guideways and bridges, and underground roof systems subject to railroad or highway loading, this code adopts the latest version of the California Building Code, California Code of Regulations, Title 24, Part 2, California Building Standards Commission, based on the International Building Code. This code and its amendments is referred to herein as the Building Code.

5.5.1 Stations, Buildings, and Framed Structures

Surface stations are defined as those stations with platforms constructed above or below adjacent finished grade (at-grade stations). Design the following structures and buildings (but not limited to the following) included in the Project in accordance with the Building Code and its referenced codes including the California Building Code Title 24, the International Building Code, and ASCE 7 when the structures do not participate in the loads carried by the aerial guideway girders.

A. All building framing and components for surface stations, excluding aerial station platforms, mezzanines, and aerial pedestrian access/ramps;

B. Maintenance facilities;

C. Ancillary facilities;

1. New building(s) by private developers representing commercial interests or other public agencies that are planning pedestrian entrance access to Metro facilities must have their designs reviewed and accepted by Metro. It is the general policy of Metro to review designs on a case-by-case basis. This includes not only plans for physical attachment but also all new construction within the influence zone of the existing Metro facilities.

2. Foundation and soils investigations and reporting requirements shall be in accordance with Section 1802 of the Building Code, except as modified herein.

3. Temporary support of project facilities during the adjacent excavation for new buildings will be such that at any level, the project facilities lateral displacement shall not exceed 0.001 times its overall height above the bottom of the base slab, but not to exceed 1/2 inches without Metro's prior approval. Unless otherwise approved by Metro in advanced and in writing, the lateral forces used for the design of temporary excavation support shall consider both the static and dynamic loads for which the project facility was designed. Temporary support shall not endanger the safety of any persons or cause damage to any property and shall conform to Section 9.0 Support and Underpinning of Existing Structures.

4. Areas of new buildings adjacent to project facilities where the public has access or that cannot be guaranteed as a secure area, such as parking garages and commercial storage and warehousing, shall be treated as areas of potential explosion. NFPA 130, Standard for Fixed Guideway Transit Systems, life safety separation criteria shall be applied that assumes such spaces contain Class-I flammable or Class-II or Class-III combustible liquids. For structural and other considerations, separation and isolation for blast shall be treated the same as for seismic, and the more restrictive shall be applied.

5. Parapets--Where parapets are used, they shall be designed to withstand dead load, wind load, force due to thermal expansion and contraction, shrinkage force, and earthquake forces equal to the full dead load of the parapet acting at the center of mass of the component parts.

6. Elevators--Surface structures shall be designed for the loads described below:

   a. Dead load of structure

   b. Live load of 100 plf applied at the free edges of the frame

   c. Wind load of 40 psf on windward side

   d. For traction type elevators, the surface structure shall be designed to support elevator beams. The end reaction of the elevator beams shall be 18,000 pounds minimum. The location of the elevator beams varies with the type of elevator and its relative machine room location. The Designer shall coordinate with elevator manufacturers regarding elevator beam locations.

D. Escalators

The support elements shall be designed for the end reactions from the escalators.

E. Elevators, Escalators, and Passenger Conveyors

Structures supporting elevators, escalators, or passenger conveyors shall be designed for the maximum reactions from any of the manufactured units considered for use in the system.

F. Stairs

Stairways shall be designed for a uniform LL of 100 psf or a concentrated load of 300 pounds on the center of stair treads, whichever is critical. Impact shall not be considered for stairways.

G. Storage Space and Machinery Rooms

Electrical equipment rooms, pump rooms, service rooms, storage space, and machinery rooms shall be designed for uniform LL of 250 psf, to be increased if storage or machinery loads so dictate. Fan rooms and battery rooms shall be designed for uniform loads of 350 psf.

H. Railings

Railings in station platforms, mezzanines and service walkways shall be designed in accordance with the Building Code.

I. Vehicular Surfaces

Gratings in areas that are subject to loading from vehicles shall be designed to carry HL-93 loading in accordance with AASHTO LRFD. Gratings in sidewalks and in areas protected from vehicular traffic shall be designed for a uniform LL of 300 psf.

5.5.2 Pedestrian Area Live Load (See also Section 5.3.4)

Pedestrian ramps, pedestrian bridges, mezzanines, and other pedestrian areas shall be designed for a uniform LL of 100 psf. Station platform areas shall be designed for a uniform LL of ~~125~~ **100** psf. Pedestrian loads shall not be subject to a dynamic load allowance.

5.5.3 Seismic Design of Buildings (See also Section 5.3.5)

Building framing and components shall be designed to resist earthquake motions in accordance with Metro Supplemental Seismic Design Criteria Appendix and the applicable codes of the Building Code. Seismic parameters shall be as prescribed by the Code or site-specific recommendations in Metro approved Geotechnical Planning Report (GPR). See Section 5.6.2.4. Note that in the context of contracting practices, GPRs are equivalent to "Draft Geotechnical Memoranda for Design". See Metro SSDC, Section 1.5.

The following structures (but not limited to the following) included in the Project shall be designed in accordance with Caltrans BDS when the structure participates in loads carried by the rail guideway girders, or in accordance with the Building Code when it does not.

- Aerial station platforms;
- Pedestrian bridges and ramps/access;
- Mezzanines.

5.5.4 Building Foundations

Building foundations shall be in accordance with Section 5.6 Geotechnical.

5.6 GEOTECHNICAL

5.6.1 Definitions

A. *Project Geotechnical Engineer* is defined herein by procurement method.

1. *Design-build (D-B)*: Design-builder's engineer of record's lead geotechnical engineer who shall be a California licensed professional engineer as defined by California Department of Commerce and Consumer Affairs (DCCA) and who shall be responsible in charge of all geotechnical work and who shall affix his stamp and seal on all project geotechnical reports. Reports shall be subject to Metro review and acceptance.

2. *Design-bid-build (D-B-B)*: Lead geotechnical engineer who shall be a California licensed professional engineer as defined by DCCA and who shall affix his stamp and seal on all project geotechnical reports and

recommendations prepared for Metro either directly or indirectly as an employee of the engineer of record or as a subconsultant to the engineer of record. Reports and recommendations shall be subject to Metro review and approval.

B. *Site* is defined per Caltrans BDS implemented AASHTO LRFD Section 10.5.5.2.3: "A site shall be defined as a project site, or portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification, i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata and the groundwater conditions." This definition is modified herein to read "contiguous portion" and not exceeding 5,000 feet in length.

C. *Dry Construction* is defined herein as the excavation condition and concrete placement method wherein the sides and bottom of shaft may be visually inspected prior to placement of concrete and where water depth at the bottom of the shaft is not more than 3 inches at the start of concrete placement and where water accumulation in the bottom of the shaft is not greater than 12 inches per hour when no water pumping is permitted.

D. *Wet Construction* is defined herein as condition not qualifying as dry construction, the excavation condition and concrete placement through water or slurry, whether intended for excavation stabilization or result of naturally occurring hydrogeologic conditions

E. *Non-redundant drilled shaft foundation* is defined herein as foundations consisting of two or fewer shafts per guideway bent or pier or those shafts deemed non-redundant per Caltrans BDS implemented AASHTO LRFD Section 1.3.4.

F. *Deep foundations* as used herein are defined to include drilled shafts, driven piles, micro-piles, and other foundation types deriving their principal support from embedment into the subsurface and where embedment depth exceeds minimum element dimension.

G. *Shallow foundations* as used herein are generally footings for which capacity is derived principally from its bearing at shallow depth below existing or final ground surface adjacent to the foundation, e.g. embedment depth generally less than foundation width or length.

Reference Caltrans BDS implemented AASHTO LRFD Section 10.2 for additional foundations-specific definitions.

5.6.2 Geotechnical Investigations, Analysis, and Design

5.6.2.1 Geotechnical Planning Report (GPR)

The lead project geotechnical engineer shall oversee preparation of a Geotechnical Planning Report (GPR) and submit the GPR to Metro for review and acceptance. The GPR defines the engineering and design approach that the designer will follow to develop the most cost-effective and technically and environmentally acceptable foundations, cut and fill slopes, retaining structures, and geotechnical designs for the aerial/bridge, underground, and at-grade portions of the project. The GPR shall define the engineering and design approach that the project geotechnical engineer will follow to develop the necessary geotechnical information for the project in accordance with the requirements of these design criteria. The GPR will address all aspects of the required geotechnical effort and foundation design and analysis, including, but not limited to, the following:

A. Succinct description of the structural and civil project components that the geotechnical work scope addresses;

B. Methods proposed to execute any of the identified investigation and data needs and develop sufficient data, including laboratory and field tests, for the analyses per Caltrans BDS implemented AASHTO LRFD Sections 10.4.3 and 10.4.5;

C. Proposed methods of analyses for the identified structural and civil components with special attention to construction methods for drilled shaft and pile foundations;

D. Proposed format of geotechnical reports and topical outline;

E. Proposed deflection criteria to be used for design of deep foundations and, where these limits exceed the limits defined in these design criteria, along with supporting documentation as justification for exceeding criteria limits.

F. In addition, the GPR will address:

- Additional Subsurface investigations;
- Determination of geotechnical design parameters;
- Determination of seismic design parameters
- Slope analysis and design;
- Embankment and fill settlement and slope stability analysis;
- Planned field testing programs;
- Ground improvement or treatment of in-situ soils;
- Selection, design and analysis of foundation systems;
- Lateral and vertical earth pressures;
- Instrumentation and monitoring programs; and
- Content and format of geotechnical reports.

G. Note that in the context of contracting practices, GPRs are equivalent to "Draft Geotechnical Memoranda for Design". See Section 5.6.2.5.

5.6.2.2 Subsurface Investigations and Laboratory Testing

The lead project geotechnical engineer shall, prior to the start of any field investigations, submit a detailed plan addressing how the planned field investigations meet the requirements of the GPR. The locations of these investigations shall be shown on a site plan not smaller than 1 inch equal to 200 feet. The plan shall clearly state the types of equipment to be used, planned completion / penetration depths, sampling types and intervals, any down hole testing planned, and completion details. In addition, the plan must address management of investigation, spoil material, maintenance of traffic requirements, environmental compliance requirement, and a time line for execution of the work, including permitting and utility clearances. Investigation methods shall conform to the recommendations of Training Course in Geotechnical and Foundation Engineering: Subsurface Investigation, Participants Manual, FHWA HI-97-021 and these criteria.

The lead project geotechnical engineer shall prepare and implement a subsurface exploration and testing program with all field and laboratory testing necessary to establish the geotechnical and environmental conditions and to provide a basis for all final geotechnical and foundation designs and analyses. The program shall be developed and implemented to supplement the data provided by Metro and to obtain data as required to support the design approach and construction methods. The project geotechnical engineer shall submit its investigation plan prior to its implementation for review. Perform the geotechnical investigation program to establish all geotechnical parameters and subsurface conditions required for design and construction.

The lead project geotechnical engineer shall prepare recommendations for foundation designs. All reports and recommendations shall be prepared and sealed by a California registered geotechnical engineer (G.E.), with experience in type of work specified on this project.

For structures subject to the jurisdiction of local authorities, the design bearing and frictional values for foundations shall not exceed the limits given by those authorities.

For underground station structures and associated appurtenant structures, geotechnical data and design parameters shall be shown on the contract drawings. The project geotechnical engineer shall investigate, at the recommendation of Metro, any other areas necessary to determine ground conditions for excavation means and methods.

Follow professionally acceptable standards, in planning, performing and reporting subsurface exploration programs. Among the requirements for the borings and laboratory investigations to be performed for the project are the following:

A. Supervision – Perform all boring and in-situ testing and all laboratory classification and testing using qualified geologists or engineers under the direct supervision of a California registered professional geotechnical engineer (G.E.);

B. Location and Ground Surface Elevation – Determine the coordinate location and ground surface elevation for each boring and field investigation and show both the Station and offset and the elevation on the Project control surveys;

C. Soil classification shall be performed in accordance with the Unified Soil Classification System; and

D. Geotechnical testing laboratory shall be certified by the City of Los Angeles.

E. For typical structural foundation investigations conducted in the state right-of-way, follow the up to date guidelines in the “Caltrans Foundation Manual”.

The field investigation programs shall include all necessary borings, soil/rock sampling, geophysical testing, or other in situ testing as needed to provide a basis for the geotechnical and foundation design to the satisfaction of Metro. Similarly, the laboratory testing program shall include all laboratory testing necessary to establish geotechnical design parameters to the satisfaction of Metro.

Submit the details of the field investigation and laboratory testing programs. Clearly present the rationale for development of the investigation and testing programs, data interpretation and input parameter selection, together with descriptions of the methods of analysis. Include a discussion of the following:

1. Variation in the subsurface conditions across the site(s);

2. Method of construction;

3. Critical combinations of loading; and

4. Other relevant factors.

5.6.2.3 Geotechnical Data Report Preparation

Prepare in the form of a geotechnical data report(s) (GDR), a summary of all geotechnical data and findings, including the results of the review of existing information, results of the field subsurface investigations, and results from the laboratory tests and geotechnical and foundation analyses and design. Include in the report:

- Project descriptions,
- Locations and results of borings,
- Geophysical testing and other in situ testing,
- Observations of groundwater monitoring wells,
- A detailed description of geological and subsurface conditions (including a description of site stratigraphy),
- A description of groundwater conditions,
- Results of laboratory tests,
- Material properties,
- Field testing,
- Chloride content, acidity (PH value) and sulfate content of the surface water, ground water, and/or soil.

All pertinent data and complete discussions of all geotechnical analyses, designs and studies, conclusions and recommendations for foundation types for structures (with appropriate design parameters) to be designed and constructed including:

- Probabilistic Seismic Hazard Analysis
- Embankments,
- Cut slopes, retaining walls,
- Ground improvement,
- Requirements for back fill materials,
- Potential groundwater problems,
- Dewatering requirements,
- Excavation support designs,
- Instrumentation and monitoring requirements,
- Grounding mat requirements;
- Potential settlement problems,
- Potential stability problems, and
- Analysis results.
- Submit the report for review and acceptance.

Incorporate Boring and in-situ test locations and information from the existing and the Contractor's investigation program into the Design and Construction Drawings as the Contractor's program proceeds.

5.6.2.4 Geotechnical Data and Baseline Reports

When appropriate and especially for underground and tunnel construction, the geotechnical investigation should be planned, executed and reported following the recommendations given in the booklet "Geotechnical Baseline Reports for Construction – Suggested Guidelines (Essex, 2007)." During the course of the investigation and design several reports may be generated but the Geotechnical Baseline Report (GBR) must be the sole geotechnical interpretive document upon which the Contractor may rely. One or more Geotechnical Data Reports (GDR) may be developed by the designer and/or the designer's geotechnical engineer and will contain the factual information that has been gathered during the exploration and design phases of the project. See the Essex publication for detailed guidance regarding the content of the GDR.

Note that the GDR is included as a contract document but that the GBR must be given precedence over the GDR within the Contract Document hierarchy. Should the GBR be silent on a given circumstance, the GDR should be reviewed to see if there is any data/information relative to the issue in question.

During the course of development of a design, the design team may need an interpretation of the geologic data before the GBR is prepared. This need may be met by one or more Memorandum for Design. To clarify its intent, use and timing in the design process, this report (or reports) should be given a title such as "Draft Geotechnical Memorandum" or "Draft Geotechnical Memorandum for Design". It must be disclosed as available information, but it is not part of the Contract Documents: it is preliminary and the interpretations and discussions presented therein will be superseded by subsequent interpretations and baselines in the GBR. Preparation of an interpretive report, such as a Geotechnical Interpretive Report (GIR), "is superfluous, a potential source of confusion, and is strongly discouraged" (Essex, 2007).

In summary, "The GBR should be the sole geotechnical interpretive document upon which the Contractor may rely. The GBR should be limited to interpretive discussion and baseline statements, and should make reference to, rather than repeat or paraphrase, information contained in the GDR, drawings, or specifications)" (Essex, 2007) see chapters 5 and 6 of the Essex document for further discussion of the suggested content and format of GBRs.

5.6.2.5 Geotechnical Designs

Project structures and improvements shall be designed so that imposed loadings do not exceed soil resistance while limiting deflections, as applicable, to prescribed maximums. Foundations supporting aerial guideways and transit rail retaining walls shall be designed in accordance with the requirements of Caltrans BDS referenced AASHTO LRFD Chapter 10 and 11, and AASHTO Seismic Guide Specification. Foundations for buildings, retaining walls, and appurtenances not governed by these design criteria, shall be designed in accordance with the Building Code as defined in Section 5.1.3, Subsection C.4. (California Building Code Chapter 18, *Soils and Foundations*). Presumptive load resistance values (i.e., maximum allowable bearing pressures and lateral resistance) shall not exceed the maximum values specified without substantiating data from the project geotechnical engineer pursuant to the above geotechnical reporting requirements. Additionally, for aerial guideway designs a minimum of 50% of the bent locations shall have been investigated in accordance with Geotechnical Investigations (listed above), as a priori to submittal of the design report required by the following subsection.

5.6.2.5.1 Deep Foundations

Design of deep foundations shall be based on project-specific information developed for the location(s) and foundation type planned. Soil and rock engineering properties shall be based on the results of field investigations as presented in the Geotechnical Data Report; use of presumptive values shall not be allowed for final design. Bottom clean out of drilled shafts constructed using the wet method shall be verified by Miniature Shaft Inspection Device® (MiniSID) or approved equal.

Designs for aerial guideway foundations shall be in accordance with Caltrans BDS. Design of deep foundations for all other structures shall be designed in accordance with the requirements of Building Code. Resistance factors for LRFD designs shall be as prescribed by Caltrans BDS and in consideration of all factors noted therein.

Where permanent steel casing is used and is relied upon for structural capacity, its thickness shall have a minimum wall thickness of 3/4 inch and be provided with internal shear lugs if composite action is to be relied upon. Additionally, the design basis of the steel section shall be reduced to account for corrosion over the life of the structure based on actual soil and ground water conditions; in lieu of a site specific corrosion study, a presumptive value of 1/4 inch shall be used. Steel casing shall not be considered for structural support in extremely aggressive environments.

Construction tolerance for all drilled shafts shall be in accordance with Caltrans BDS. For guideway shafts greater than 5 feet in diameter, the drilled shafts shall be designed assuming they are offset at the top of the shaft a minimum of 6 inches. For further detailed information see Standard Specification Section 31 63 30 - Drilled Concrete Shaft Foundations.

Load tests shall be performed on a minimum of one drilled shaft at any given Site but not less than one per 5,000 contiguous feet or portion thereof of aerial guideway alignment where subsurface conditions are defined as being similar (i.e., Site) in the Geotechnical Data Report. See Section 5.6.2.3.

Design of mini-piles shall be in accordance with Caltrans BDS (AASHTO LRFD Section 10.9 Micropiles) and FHWA-SA-97-070 (Micropile Design and Construction Guidelines, June 2000).

Tops of deep foundations, including top of drilled shafts or pile caps where multiple shafts or piles are employed, shall be a minimum of 2 feet below lowest adjacent finished grade.

The upper 5 feet as measured from lowest adjacent grade shall be discounted in any axial and lateral load analyses except where it can be shown that measures are provided to prevent future excavations around the pile within three diameters from the shaft or pile group exterior surface.

5.6.2.5.2 Shallow Foundations and Miscellaneous Structures

A. Shallow Foundations

Per Caltrans BDS (AASHTO LRFD Section 10.2 Definitions): “Shallow Foundation— A foundation that derives its support by transferring load directly to the soil or rock at shallow depth.”

Design of shallow foundations, e.g., spread and strip footings, shall be based on project-specific information developed for the location(s) and foundation type(s) planned. Soil and rock engineering properties shall be based on the results of field investigations as presented in the Geotechnical Data Report; use of presumptive values shall not be allowed. Designs of shallow foundations supporting rail structures or attached appurtenances shall be as required in Caltrans BDS (AASHTO LRFD Section 10.6) and in accordance with FHWA-SA-02-054 (Geotechnical Engineering Circular No. 6 Shallow

Foundations)). Shallow foundations for support of structures under the purview of the Building Code, buildings not directly supported off the aerial guideway, shall be designed in conformance with the requirements of the Building Code, Section 1805 (Footings and Foundations) Shallow foundations shall have a minimum ground cover of 2 feet as measured from top of footing to finished grade.

B. Miscellaneous Structure Foundations

Design of foundations for miscellaneous structures shall be in accordance with the above requirements for shallow foundations, excepting that presumptive values may be used. These include, but are not limited to miscellaneous structures such as light standards, signs, retaining walls less than 10 feet in height and not supporting any structures, and other such lightly loaded and uninhabited structures.

5.6.2.6 Settlement and Deflection

Allowable foundation settlements and lateral deflections (deformations), except as prescribed herein, shall be established by the project structural engineer in consultation with the project geotechnical engineer.

5.6.2.6.1 Deep Foundations

Settlement of deep foundations (i.e., drilled shafts or driven piles) shall be limited to no more than 1/2 inch total vertical deflection as measured at the pile head or top of pier cap after placement of the pier. Total settlement measured after placement of the guideway girder shall be limited to not more than 1 1/2 inches. Differential settlement between adjacent bents spaced not more than 100 feet apart shall be limited to no more than 1 inch; this maximum decreases proportionately for lesser bent spacing and increases by ½ inch per 100 feet for bent spacing exceeding 100 feet.

Lateral deflection limitations for design of deep foundations for non-seismic loading shall be determined by the project structural engineer and systems engineer in consultation with project geotechnical engineer. Deflections of deep foundations under extreme or earthquake loadings shall be established by the project structural and geotechnical engineers but not greater than the deflection and rotation which would result in a deflection of 18 inches at the top of rail.

5.6.2.6.2 Shallow Foundations

Shallow foundations shall be designed to limit total settlement to no more than 1 inch and differential settlements to no more than 1/2 inch.

5.6.3 Soil-Structure Interaction

5.6.3.1 Approach to Structural/Geotechnical Design Problems

Soil-structure interaction is a stress-strain issue in the mechanics of both structural and geotechnical materials. The investigation of this subject shall be conducted for the design of both underground structures and the foundations of bridges and aerial guideways. To arrive at a design solution for a structure whose structural elements interface with the ground's soil media, the pressures and distortions at their interface shall be demonstrated to be compatible. The final analytical solution shall include all the variable, static and dynamic forces that are imposed on and impact both the structure and its surrounding ground media by this Structural/Geotechnical Criteria.

Either one or both of two rational approaches shall selected and used to assist in arriving at the practical engineering design and the safe and economic construction of underground structures and the foundations of bridges and aerial guideways. The appropriate analytical choice for design depends on the final GPR and the actual planned construction together with the collective judgment of the structural and geotechnical engineers involved. Some guidelines to be considered are offered here.

Approach 1 Subgrade Reaction Springs Coupled with the Finite Element Structural Model

The material stress-strain solution discussed here is the use of subgrade reaction in the analysis of soil-structure interaction problems expressed very simply as:

$$k_S = p/S$$

Where:
k_S = the subgrade reaction in pounds per cubic inch
p = pressure on an element such as a foundation in pounds per square inch
S = the corresponding settlement in inches

Subgrade reaction is represented in analysis as a linear spring element intended to duplicate the same response as the Modulus of Elasticity of the ground would in the actual structure being designed. A finite element model of the structure is produced and the subgrade reaction soil springs intended to represent the response of the soil to movement of the structure are attached to node points on the structure model. A substantial amount of trial and error analysis is required in arriving at a compatible structure-ground solution, and it takes experienced designers to understand the impact of various structural configurations and directions of movement in the ground media (for example vertical versus horizontal & etc.) have on the value of the subgrade reaction.

Approach 2 Finite Element or Difference Analysis of a Continuum of the Ground Media's Soil Modulus of Elasticity Coupled with the Structural Model

In this case E_S and μ are derived from the geotechnical investigation and applied as parameters in the finite element or finite difference program itself.

Where:

E_S = Stress-strain modulus or modulus of elasticity of the soil in pounds per square inch
μ = Poisson's ratio = Strain perpendicular to the applied stress/strain in direction of applied stress

A finite element or finite difference model of the structure and its surrounding soil continuum, and any nearby underground or surface structures or other features impacting the results of the analysis are modeled simultaneously.

There are times when this approach is the most appropriate, such as estimating the surface settlement due a tunnel being bored below, or to estimate the influence on the ground due to a bored tunnel following behind and closely adjacent to one already bored. Also, the design of mined tunnels and stations require this approach in a design using sequential support of excavation. Another example might be the deep caissons of a major bridge crossing or the complex ground conditions being encountered by a long submerged tube tunnel where such detailed analyses are anticipated by the client.

The problem arises from the black box nature of complicated finite element programs. The designer shall verify the accuracy of such methods to the satisfaction of the Metro by a written report and with calculations that explain the theory, the input values, and the results. The designer shall also verify that the person writing the program actually understands the intricacies of every problem; and that the person providing the stress-strain parameters for computation understands which bracketed material values will produce the most critical results. For quality assurance verification the designer shall demonstrate the validity of the results of the analysis using less complex, more visually understandable analytical comparisons.

5.6.3.2 The Design of Cut and Cover Stations

For the purpose of the scope of these criteria, Approach 1, the use of subgrade reaction springs is judged to be the more appropriate design method, especially for major cut and cover construction such as those for station shells. The structural configurations are complex, involving multiple levels of station with varying depths of cover and methods of temporary and permanent ground support. Station shells are complicated by end walls and their interfaces with tunnels, and by ventilation shafts, entrances, and other appurtenant facilities intersecting with the shell whose proportions vary and are subject to varied construction sequencing throughout the station’s length. Also, loading conditions require that both elastic and ductile plastic distortions must be accounted for during the life of the structure. In addition, the process of design covers conceptual, preliminary, and final design phases in which operational, passenger access, and architectural planning decisions require frequent alterations to the structural/geotechnical configuration of the station making the frequent repetition of the long process presented by Approach 2 impractical.

Due to the uncertainty of obtaining accurate values for the elastic properties of the ground, a sensitivity analysis of the intended cut and cover station structure shall be conducted in determine the cost of accommodating the range of soil values needed for accurate analysis. An intelligent design that does not require precise soil data to be economic shall be used where practical for construction. An example of this is employing excavation struts into the final structure rather than temporary struts that allow additional loss of ground movement when removed during construction.

5.6.3.3 The Design of the Foundation for Bridges and Aerial Guideways

A simplified application of Approach 1 can be used for the design of regular footings for the piers and abutments of bridges and aerial guideways. This approach can also be extended to multiple pile footings where the piles act mainly in tension and compression. For deep foundations using a single large diameter drilled shaft, or a combination of drilled shafts, where the lateral distortion of the piles takes an active role in the response of the foundation to static and dynamic loading, either a more refined application of Approach 1 should be used or Approach 2 should be implemented.

5.6.3.4 Geotechnical Investigation and Testing

Several methods are available for determining the geotechnical stress-strain data used by both Approach 1 and Approach 2:

- Unconfined compression tests
- Triaxial compression tests
- In-situ tests
  - Standard penetration tests
  - Cone penetration tests
  - Pressuremeter
  - Plate-load tests

Test results shall be evaluated by a qualified geotechnical engineer, but also values based only on the observation of ground conditions by the geotechnical engineer are an acceptable approach to conceptual and preliminary design.

5.6.3.5 Load Effects on Soil Modulus and Subgrade Reaction Characteristics

The designer's prediction of the soil resistance at any point along a structural element subjected to loading shall address the stress-strain characteristic of the soil. The characteristics are shared by problems that involve values of both K_S and E_Y. These properties shall be those existing after the structural element has been installed. Construction has an especially significant influence on clayey soils. In addition, there are four classes of loading that shall be considered:

Static Loading – Static loads shall be considered as short term and not repeated; and sustained loads where the soil is not susceptible to consolidation and creep. (For example, over consolidated clays, soft rocks, and clean sands)

Sustained Loading – The effect of sustained loads shall be considered. These are the most common condition for the design and construction of underground structures. If the soil resisting sustained loading is granular material that is freely-draining, the creep can usually be assumed to be small. If the soil is soft, saturated clay, the stress applied to the soil by the structural element can cause a considerable amount of additional deflection and a reduction in the effective value of the soil modulus or subgrade reaction. An example of a sustained load that cannot be reversed is the removal of soil from a deep excavation. The elastic and plastic relief at the bottom of the trench causes the ground to rise where it is excavated causing a permanent reduction in the soil modulus or subgrade reaction.

Cyclic Loading – The factors that shall be considered for structures subjected to cyclic loading are the frequency, magnitude, duration, and direction. Some of the cases are wind load and the live load and impact of rail vehicles on the foundations of aerial guideways. In the case of deep underground stations, the construction equipment being used for excavation can cause a cyclic reduction in the effective value of the soil modulus or subgrade reaction at the bottom of the excavation.

Dynamic Loading – The effect of dynamic loading shall be considered for structures such as machine foundations and earthquakes. The deflection from the vibratory load of a machine foundation is usually small and can be solved using the dynamic properties of the soil. For earthquake loading, a rational solution shall proceed from the definition of the free-field motion of the soil due to the earthquake. Seismic free-field motion is discussed in the Appendix of this criteria.

5.6.3.6 Values of Soil Modulus and Subgrade Reaction for Conceptual Design

Soil modulus and subgrade reaction values for final design shall be provided by the Geotechnical Planning Report (GPR) and substantiated during design by the project geotechnical engineer. A geotechnical engineer shall also approve the use of preliminary values used for analysis presented in this section of the criteria. The values shall b consistent with the GBR.

The designer shall use conservatism in the obtaining and testing of soil samples, and the safe application of the resulting values. A comparison with concrete sampling and testing can be made. In spite of the high degree of control in making, sampling, and testing structural concrete, the American Concrete Institute recommends the Young's Modulus for concrete should not be anticipated to be more accurate than plus or minus 20 percent of the test value. Therefore the designer shall develop a structural solution that does not require the application of values not acceptable as conservative by the Metro.

Figures 5-9 and 5-10 Explain the definition of Soil Modulus (E_S) and Subgrade Reaction (k_S) as used by this criteria. The Soil (Stress-Strain) Modulus can be obtained from the slope (tangent or secant) of stress-strain curves from triaxial tests.

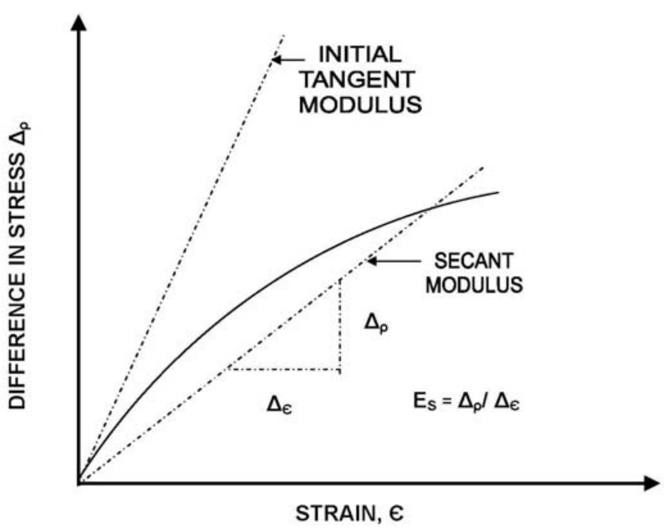


FIGURE 5-9 SOIL MODULUS Es

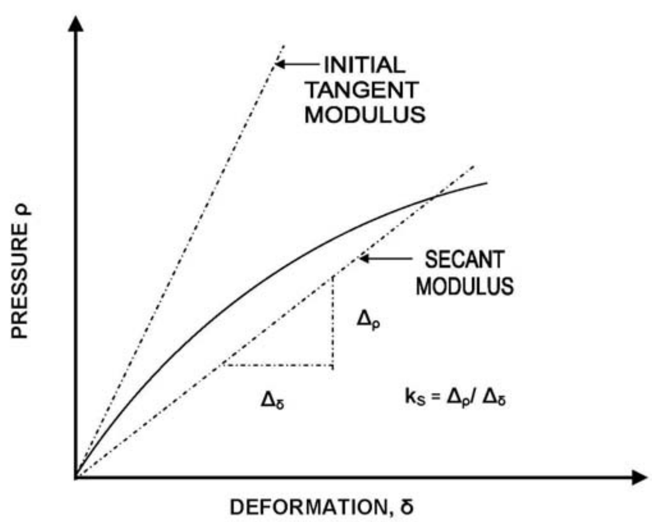


FIGURE 5-10 SUBGRAGE REACTION ks

Subgrade Reaction k_S is the ratio of stress to deformation and typical values are provided by Table 5-6. These values can be viewed as those produced by a rigid plate of about one square foot in area in the undisturbed earth approximately at a depth immediately under a spread footing. The problem for immediate conceptual work for an underground structure or a deep foundation is that for most soils, k_S increases with depth. See the discussion under Section 5.6.3.7.

TABLE 5-6 TYPICAL RANGE FOR SUBGRADE REACTION k_S

SOIL	ks, kips/cu ft
Clayey soil	
$q_u \leq 4$ kips/sq ft	75-150
$q_u \leq 8$ kips/sq ft	150-300
$q_u \leq 16$ kips/sq ft	> 300
Where q_U = the unconfined compressive strength	
Sand	
Loose	30-100
Silty medium dense	150-300
Clayey medium dense	200-500
Medium dense	60-500
Dense sand	400-800

The most available information at the onset of a project is that provided by soil borings of other nearby projects or the initial field borings. The most available information is therefore the standard penetration test (SPT) blow count or the cone penetration test (CPT) cone resistance. Table 5-7 provides some of the approximate correlations between these values and Soil Modulus (E_S) that may be related to soil values at increased depths.

TABLE 5-7 SOIL MODULUS E_S by COMMON TEST METHODS, kps (or units of q_C)

SOIL	SPT	CPT
Sand (normally consolidated)	$E_S=10(N + 15)$ $E_S = (300 \text{ to } 440) \ln N$	$Es = (2 \text{ to } 4)q_C$ $E_S = (1+D_R^2)q_C$
Sand (saturated)	$E_S = 5(N+15)$	
Sand (overconsolidated)	$E_S = 360 + 15N$	$E_S = (6 \text{ to } 30)q_C$
Gravelly sand and gravel	$E_S = 12 (N+6)$ $N \leq 15$ $E_S = 12 (N+6) + 2000$ $N>15$	
Clayey sand	$E_S = 6(N+15)$	$E_S = (3 \text{ to } 6)q_C$
Using the Undrained Shear Strength S_U in Units of S_U		
Clay	$I_P > 30$ or organic $I_P < 30$ or stiff	$E_S = (100 \text{ to } 500)S_U$ $E_S = (500 \text{ to } 1500)S_U$

Where:
- SPT = Standard Penetration Test
- CPT = Cone Penetration Test
- N = SPT blow count
- q_C = CPT cone resistance
- D_R = Relative density
- I_P = Plasticity index = liquid limit – plastic limit

The conceptual value of k_S can then be derived for the estimated value of E_S and an educated approximation of Poisson's ratio for the soil in question.

The relationship between k_S and E_S is:

$$k_S = E_S/B(1-\mu^2) \text{ [Joseph E. Bowles]}$$

where:

B = the characteristic footing width, wall height, or deep foundation width where everything is inconsistent units. (See Figure 5-11)
μ = Poisson's ratio

From Figure 5-11, if "U" is small, say 1-foot or less, the rigid beam length "B" sitting on an elastic foundation would normally be considered a 2-dimensional problem. Nevertheless, if the beam is intended to represent a footing the length of NxU, it is actually a 3-dimensional problem whether the ground continuum under the footing is represented by the value k_S or E_S. Using the value E_S in a 2- dimensional finite element program that includes a large segment of the ground underlying the footing is therefore only marginally better than using a simple spring representing k_S. Both require the geotechnical engineer to develop a rationale that corrects for misrepresenting the 3RD ground dimension.

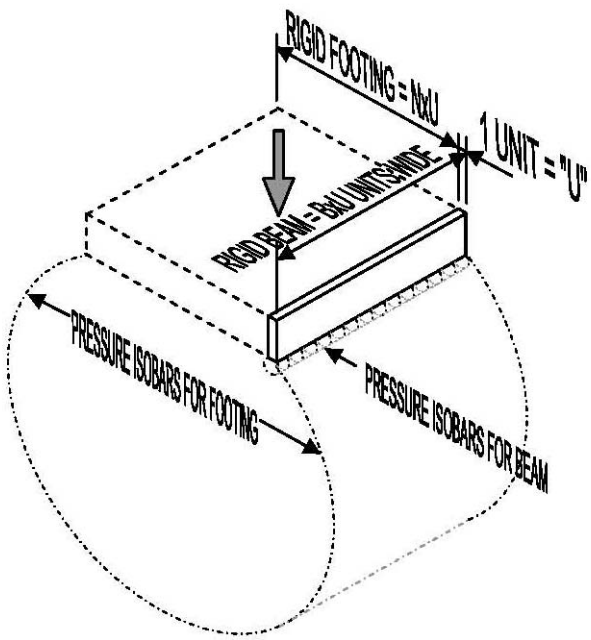


FIGURE 5-11, THE PROBLEM OF 2-DIMENSIONAL APPROXIMATIONS OF E_S & k_S

The finite element solution is also more cumbersome to set up and use for something as complex as a cut and cover station with numerous cross-sections that must be ~~analyzed~~ **analysed** through several adjustment cycles and each with numerous ranges of soil conditions and construction sequences. Using soil springs to represent k_S makes the solutions compatible with elastic beam theory and there fore both numerically and visually apparent both for their elastic and plastic components. We know by observation, for example, when a soil spring is attempting to go into tension, an impossible occurrence.

TABLE 5-8, APPROXIMATIONS OF POISSON'S RATIO

TYPE OF SOIL	μ
Clay, saturated	0.4 – 0.5
Clay, unsaturated	0.1 – 0.3
Sandy clay	0.2 – 0.3
Silt	0.3 – 0.35
Sand, gravelly sand	- 0.1 – 1.00
(common values	0.3 – 0.4)
Loess	0.1 – 0.3
Rock	0.1 – 0.4
Concrete	0.15

5.6.3.7 Characteristic Configuration Effects on Subgrade Reaction

As given above, B is the characteristic footing, wall or deep foundation dimension. Also above, the values for k_S provided by Table 5-6 are those on a small square plate in the undisturbed earth approximately at a depth immediately under a spread footing. The value of k_S changes radically with the size and shape of the structural element being loaded.

In applying a value to k_S for design the purpose of analysis, the designer shall take the characteristics of the size and configuration of a structural element in determining the appropriate value of k_S for conceptual and preliminary design and shall use only values recommended by the GPR. See Section 5.6.3.6. An example of the effects of configuration and size is given below in Figure 5-11 for a shallow square or continuous footing.

If one were to judge that the hypothetical settlement soil column height for computing k_S for a square footing were the characteristic dimension B (using the 0.3 isobar), then the value of k_S at the center of the plate would be about 0.25/.35 = 0.71 times the value at the edge of the plate. For a continuous footing k_S would be about 0.24/0.28 = 0.86 times the value at the edge of the plate.

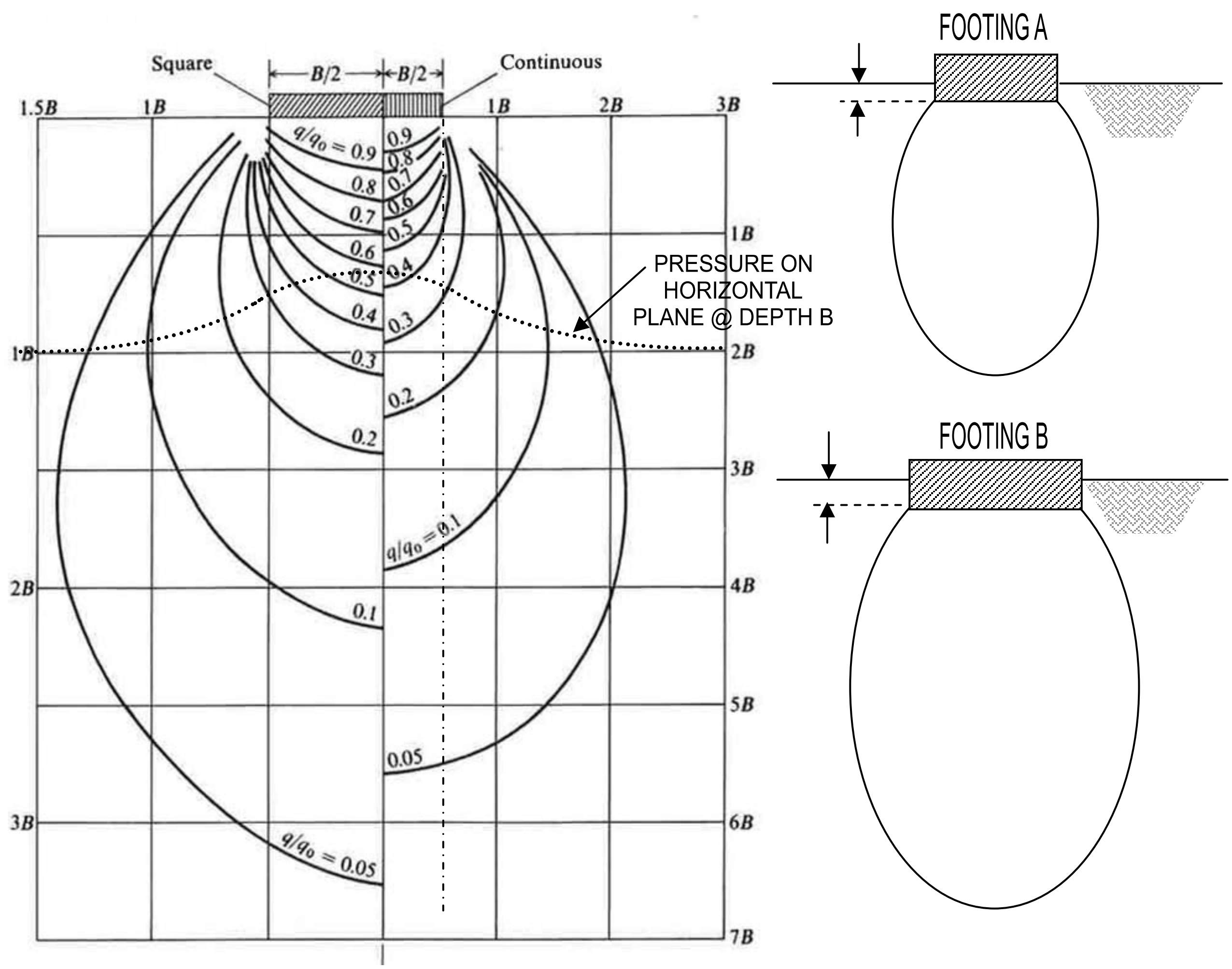


Figure 5-12, Pressure Isobars Under a Footing Based on Boussinesq

Likewise, it is well known that a square footing such as Footing B will settle more than Footing A under the same uniform pressure and underlain by the same homogeneous soil material. Conversely, in making a settlement analysis based on the k_S value obtained from a 1-foot square testing plate, the larger the footing being ~~analyzed~~**analysed**, the greater the reduction in value of k_S that must be made.

For underground structures and deep foundations, the same configuration and size effects occur regardless of orientation of the structural element. That is, the configuration of a wall influences k_S the same way that a footing does.

The designer shall also consider the effects on k_S due to excavation for a deep cut and cover structure. Expansive underlying soils and soils subject to cyclic loading can both reduce the support value anticipated from the subgrade reaction. See Figure 5-13.

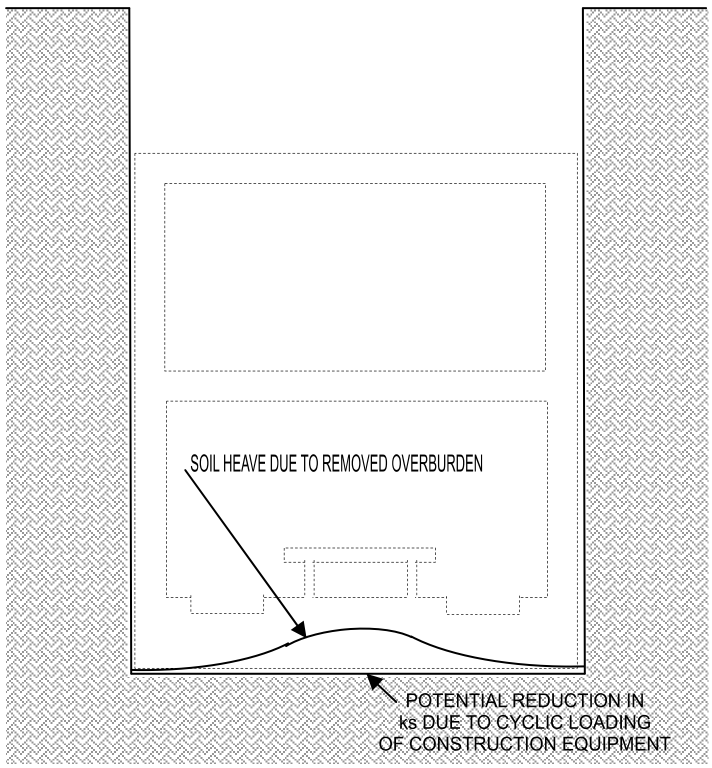


Figure 5-13 Deep Excavation for a Cut and Cover Station

5.6.4 Support of Excavation Structures

A. Contract drawings and specifications shall cover traffic diversions, mandatory restrictions, and necessary construction staging by public authorities and utility companies as applicable. Acceptable locations for construction access ramps, or any other construction facility that affects the work shall also be indicated.

B. Detailed design of the temporary decking, sheeting, and bracing shall be prepared by the contractor and reviewed by Metro, based upon criteria and design standards included in the contract drawings and specifications.

C. The Metro designer shall perform a conceptual preliminary design of decking, sheeting, and bracing utilizing the criteria that will appear in the contract documents. The design shall be for the purposes of evaluating the support of excavation system associated with the underground situation, for determining the need for supplementing or revising the criteria, and for arriving at a cost estimate for decking, sheeting, and bracing.

D. The designs shall not be shown in the contract documents except to the extent necessary to clarify unique situations not adequately addressed by the written criteria. In any event, detail design of decking, sheeting and bracing shall not be shown.

E. It shall be a requirement in the contract documents that the design of support of excavation structures shall be prepared by an engineer registered in the state of California. The review and acceptance of the designs submitted by the contractor shall be made by an engineer registered in the state of California.

5.6.5 Support and Underpinning of Existing Structures

5.6.5.1 General

A. The lead structural/geotechnical engineer shall investigate all buildings and structures to remain over, or adjacent to and within the influence zone of the construction, the work, and prepare all necessary designs for their protection during construction and/or permanent support and underpinning. Review by the project geotechnical engineer is required.

B. All buildings or structures to be considered include, but are not limited to, the following:

1. Buildings or structures that extend over the transit structures to such an extent that they must be temporarily supported during construction and permanently underpinned or otherwise supported.

2. Buildings or structures immediately adjacent to the transit structures that must be carried on underpinning. These are to be braced to act as retaining elements supporting the sides of the excavation.

3. Buildings or structures that may be affected by groundwater lowering: In certain areas, lowering of the groundwater for rail transit construction may cause settlements of buildings either adjacent to or some distance from the cut-and-cover or tunneled excavation. Evaluate potential settlement and design protection for adjacent structures.

4. Any other buildings or structures for which the Metro states that it is appropriate for the designer to prepare designs.

C. The lead geotechnical engineer shall evaluate each building which could be influenced by project construction. Allowable settlement or distortion of a building varies depending on the building's construction, age, materials, use, and other factors. Thus, these building must be evaluated on a case by case basis. As a minimum however, design and construct to limit settlements to less than ½ inch, and angular distortion to 1/600.

D. Extend underpinning walls or piers supporting buildings or structures and forming a portion of the excavation support system to a minimum depth of 2 ft-0 in. below sub grade elevation of the underground structure, or to sound bearing material, whichever is greater.

5.6.5.2 Methods

Methods used to underpin or protect these buildings or structures depend on local soil conditions and may include the following:

A. Pier, Pile, or Caisson Method of Underpinning

If soil conditions, structure size, and proximity to the underground structure dictate underpinning piers, piles, or caissons, extend piles or piers below an elevation determined from sloped line drawn from the side of the excavation at a point 2 ft- 0 in. below subgrade elevation to the intersection with the vertical projection of the underpinned building foundation, or to sound bearing material whichever is greater. Determine the slope of this line in consultation with the geotechnical engineer.

B. Protection Wall Method of Structure Protection

Under some soil conditions, the supporting system for the excavation will be sufficient to protect light structures. Under heavier loading conditions, a reinforced concrete cutoff wall, constructed in short clay-slurry-filled (slurry wall) trenches or bored pile sections braced with preloaded struts, could be considered as an alternative to underpinning or to avoid settlement due to dewatering.

C. Stabilization of Soil

In general, techniques such as freezing and chemical injections for the stabilization of soil under buildings in lieu of underpinning shall not be used, except when the underground structure is directly under the building to be protected. Design building protection using soil stabilization/grouting methods in after consultation with the Metro.

D. Temporary (initial support) Bracing Systems

A tight bracing system is required to minimize temporary support movement. In addition to any requirements for support of excavation that are provided on the contract drawings, design special requirements for the installation and removal of the temporary bracing systems that relate to the designs of underpinning and protection walls, such as the levels of bracing tiers, the maximum distances of excavation below an installed brace, and the amount of preloading.

5.6.6 Hazardous Materials Investigation and Analysis

The contractor shall provide a Site Assessment study for the selected alignment(s). A site reconnaissance shall be performed to observe surface conditions, access limitations, and current activities along the proposed alignment(s). An inventory of potential contaminant sources on and adjacent to the right-of-way shall be completed based upon visual observations. A record review shall be performed using, but not limited to, historic photographs, fire insurance maps, and business directories to characterize the past activities along the alignment. To supplement information gathered from records review, the contractor shall meet with regulatory agency staff and other persons having knowledge and usage of past sites and adjacent or surrounding property.

The contractor may be asked to perform additional studies as supplemental tasks to the base scope. Subsurface investigations are to include laboratory testing for environmental criteria – hydrocarbons and metals are of primary concern. Gas, soil, and groundwater samples are necessary. Other related tasks shall include, but are not limited to, identifying and recommending mitigation measures, seeking site closures from the affected jurisdictional agency, acquiring Regional Water Quality Control Board permits for the discharge of groundwater and potable water, studying aerially-deposited lead (ADL), performing lead paint and asbestos surveys for any buildings to be demolished, and investigating construction air quality impacts to schools, day care centers, and hospitals. If any of these investigations require entry onto private property, the contractor shall provide detailed information regarding the planned work, and Metro will seek permission. Any mitigation identified as part of the above investigations shall be included in the cost estimate. Results of above investigations and testing shall be included in the Report.

5.7 CONSTRUCTION INSTRUMENTATION MONITORING PROGRAM

The designer shall draw on Metro's standard specifications and drawings to prepare contract specifications and drawings for construction instrumentation and monitoring to be implemented by the contractor. The contractor is responsible for furnishing, installing, maintaining, monitoring and removing geotechnical instrumentation for the proposed tunneling and excavations as indicated. The contract specifications shall provide for the action levels at which corrective measures are required by the contractor.

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END OF STRUCTURAL/GEOTECHNICAL

APPENDIX

METRO SUPPLEMENTAL SEISMIC DESIGN CRITERIA

METRO SUPPLEMENTAL SEISMIC DESIGN CRITERIA
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CHAPTER 1

INTRODUCTION

1 Introduction

This Seismic Design Criteria Revision updates the latest documents prepared in 2003 for Metro Gold Line Eastside Extension, and is compatible with the revision to the Metro Design Criteria Section 5 references to seismic design of structures, and provides an update to Section 5 Appendix Chapter 3 Part A for Aerial Guideways and Bridges, and the addition of Chapter 3 Part B for Underground Structures.

1.1 Background

In 1981, Southern California Rapid Transit District (SCRTD), the agency responsible for the design and construction of the Metro Rail project in Los Angeles retained Converse Consultants, the general geotechnical consultant and study team of special geotechnical experts to develop reasonable seismic design criteria for the proposed 18 mile segment of the project.

In May 1983, a report titled “Seismological Investigations & Design Criteria” was published. Part I of the report included a comprehensive review and evaluation of available geologic and seismologic information, determination of probable ground motion along the proposed route, estimation of representative 100 year probable and maximum credible ground motions and response spectra for the project. Part II of that report provides guidance and criteria to be used for seismic design. Appendix A, Part II of that report provides general discussion on the seismic design approach and philosophy, defines seismic classes, and details for the structural design. Appendix B of that report is titled “Commentary” and contains an expanded discussion of items covered in Appendix A.

In June 1984, Metro Rail Transit Consultants, general consultant to SCRTD, published “Supplemental Criteria for Seismic Design of Underground Structures”. This document has provided structural seismic design criteria for underground structures on past Metro Rail projects. Those criteria provide step by step procedures and figures to determine earthquake imposed deformations (racking) within different geologic units for Operating Design Earthquake (ODE), and Maximum Design earthquake (MDE) and structure mechanisms for the acceptable conditions during MDE.

The above referenced reports show commendable research and scholarship, which made them the “state of the art” for the seismic criteria for underground structures. The major principles espoused in these reports stood the test of the time and are used elsewhere around the world.

After the 1989 Whittier Narrows Earthquake and 1994 Northridge Earthquake, Engineering Management Consultants, general consultant to MTA, retained Woodward Clyde Consultants to prepare: 1) complete Probabilistic Seismic Hazard Analysis (PSHA) for each of the four planned Eastside stations and 2) develop representative response spectra based on PSHA results, for the Eastside Extension (The underground alignment which was subsequently abandoned). Woodward Clyde Consultants recommended adopting racking and horizontal and vertical accelerations for ODE and MDE rather than using the figures from the earlier supplemental criteria. The recommendation to change the seismic criteria for underground stations was implemented by MTA in 1997.

In the preparation of the design-build performance specifications for the Metro Gold Line Eastside Extension produced Section 01152.05, Appendix A, Structural/Geotechnical Supplemental Criteria for Design of Aerial Structures and Bridges, and Appendix B, Structural/Geotechnical Underground structures. Updating the Metro Design Criteria Section 5 references to seismic design of structures, Section 5 Supplement A for aerial structures, and the addition of Supplement B for Underground structure are the current revisions described in the Metro Supplemental Seismic Design Criteria documented by this Report.

1.2 Two Level Approach to Seismic Design

The choice of the design ground motion level, whether based upon probabilistic or deterministic analysis, cannot be considered separately from the level of performance specified for the design event. Oftentimes, important facilities are designed for multiple performance levels (e.g., with a different ground motion level assigned to each performance level, a practice referred to as performance based design. Common performance levels used in design of transportation facilities include protection of life safety and maintenance of function after the event. A safety level design earthquake criteria (a “rare” earthquake) is routinely employed in seismic design. Keeping a facility functional after a more frequent earthquake adds another requirement to that of simply maintaining life safety, and is typically only required for important facilities.

Current AASHTO LRFD Design Specifications and Guide Specifications has no explicit requirements for checking bridge performance for more frequently occurring ground motions than those that occur every thousand years, on average. But many owners want to be assured that certain important bridges will be functional in frequently occurring earthquakes such as those with return periods of the order of a hundred years or so.

Since Metro Rail is a very important transit facility that requires substantial financial investment and has significant economic consequences if it fails, a two-level ground motion approach to seismic design similar to that outlined in Applied Technology Council/Multidisciplinary Center for Earthquake Engineering Research, 2003, *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges* (ATC-49) is appropriate. The Maximum Design earthquake (MDE) and the Operating Design Earthquake (ODE) discussed below form the basis of the two-level ground motion approach adopted for the Metro Rail project.

The Maximum Design earthquake (MDE): The collapse or significant disruption of the Metro Rail system during or after a major seismic event could have catastrophic effects not only on the Metro Rail system itself but also on many other aspects including the potential disturbance to other surface structures above the collapsed underground tunnel structures and the direct and indirect business and social losses. Furthermore, the repair to or replacement of an underground structure (which forms a major portion of the Metro Rail system) is considerably more difficult and costly than that for surface structures such as buildings. Modern buildings are being designed to withstand seismic ground motions with a return period of approximately 2,500 years. The risk for the Metro Rail structure collapse needs to be at least no greater than that for the buildings. Many recent transit and important transportation facilities have also adopted the 2,500-year criteria for the safety level ground motions, including the Seattle Sound Transit Bridges and Tunnels, the Seattle Alaskan Way Tunnels, the New York City Transit Bridge and Tunnels, and the New Jersey Transit (Bridges and Tunnels). Therefore the Metro Rail structures need to be designed to sustain seismic ground motions based on the 2,500-year criteria (i.e., 4% exceedance in 100 years). The Metro Rail structures should meet the life safety performance level (no collapse) discussed above. The service level would allow disruption to general traffic, but some limited access for light emergency vehicles should be available. Given the difficulty with abandoning or replacing a transit facility of this size and nature, a repairable damage level should be considered in lieu of “significant damage” sometimes used for other projects.

Operating Design Earthquake (ODE): In practice, where a lower level (more frequently occurring) earthquake is chosen to check functionality, the selected return period has varied from project to project, even within the same geographic region. In the west coast (e.g., cities in California and Seattle), a return period typically in the range of 100 to 150 years have been used for various transit projects (e.g., 108 years for the Seattle Alaskan Way Tunnel, and 150 years for the Seattle Sound Transit and the SF Central Subway). The lower-level design earthquake selected used for these projects is one that is expected to occur during the service life of the facility, typically based on a 50% chance of exceedance in the life of the facility. Since the design service life of the Metro Rail is 100 years, the corresponding return period for a 50% chance of exceedance is about 150 year. Therefore for the lower level design earthquake (i.e., the ODE) a return period of 150 years (50% probability of exceedance in 100 years) is selected for the Metro Rail project. One of the primary purposes in designing for the lower-level ground motions is to reduce the likelihood of future repair and maintenance costs by minimizing damage during more frequently occurring earthquakes. The service level requirement under the ODE is for the facility to be put back in service for general traffic immediately after a post-earthquake inspection. This applies not only to the structure but also to the mechanical systems needed for safe tunnel operation. The damage service level is none to minimal.

The procedures to develop the MDE and ODE ground motion criteria for Aerial and Underground Structures are described in Chapter 2.

1.3 Design Policies and Objectives

The criteria and codes specified herein shall govern all matters pertaining to the design of Metro owned facilities including bridges, aerial guideways, cut-and-cover subway structures, tunnels, passenger stations, earth-retaining structures, surface buildings, miscellaneous structures such as culverts, sound walls, and equipment enclosures, and other non structural and operationally critical components and facilities supported on or inside Metro structures. These criteria also establish the design parameters for temporary structures. The minimum design life objective for permanent structures designed to meet this criteria shall be 100 years.

These seismic criteria also apply to existing adjacent buildings, their foundations, and their utility services not owned by Metro, but that fall into the zone of influence of Metro’s temporary and permanent facilities being designed. Where cases of special designs are encountered that are not specifically covered by these criteria, the designer shall bring them to the attention of Metro to determine the technical source for the design criteria to be used.

1.3.1 Design Policy

Metro Rail projects are large-scale public projects in areas susceptible to major earthquakes. Earthquake initiated failures of associated structures and systems could lead to loss of life and/or major disruption of transportation systems.

The philosophy for earthquake design for these criteria is to provide a high level of assurance that the overall system will continue operating during and after an Operating Design Earthquake (ODE). Damage, if any, is expected to be minimal and to minimize the risk of derailment of a train on the bridge at the time of the ODE. Further, the system design will provide a high level of assurance that public safety will be maintained during and after a Maximum Design Earthquake (MDE).

1.3.2 Design Objectives

For the ODE, which may occur more than once during the normal 100 year life expectancy, the structure should be designed to respond without significant structural damage; the low level of damage that may occur shall be repairable during normal operating hours.

For the MDE, which has a low probability of being exceeded during the normal 100 year life expectancy, the structure should be designed to survive the deformation imposed, avoid major failure, and maintain life safety. The objective is to provide adequate strength and ductility to prevent collapse of the structure. The extent of the structural damage should be limited to what is visible and repairable.

<u>Aerial Guideways and Bridges</u> -- In the case of bridges and aerial guideways, the design shall not result in less seismic performance capability than that required by Caltrans. To substantiate that this necessity has been met, design check calculations using Caltrans criteria may be required. The foundations of bridge and aerial guideway associated structures shall be designed taking into account the effects of soil-structure interaction. The American Disabilities Act requirements between the vehicle floor and station platforms will be considered in the analysis of dead and live load deflections and camber growth. The full loads resulting from construction equipment and other temporary elements shall be applied unless otherwise allowed by Metro. Detailed Seismic Design Criteria are documented in Chapter 3, Part A.

<u>Underground Guideway and Structures</u> -- For the seismic design and analysis of underground tunnels and support spaces circular in section, the structures should be based primarily on the ground deformation as opposed to the inertial force approach. In cases where the underground structure is stiff relative to the surrounding ground, the effect of soil-structure interaction shall be taken into consideration. Other critical conditions requiring soil-structure interaction verification include the contiguous interface between flexible and rigid components or the interface of two different structures such as a tunnel and a station, a cross-passage or ventilation building, and a station and an

entrance, or a vent shaft. Detailed Seismic Design Criteria are documented in Chapter 3, Part B.

Ancillary Surface Facilities – Some ancillary facilities are subject to both the code forces normally applied to surface buildings as well as those being applied to the transit guideways. Whichever code applies the most critical set of requirements shall apply to the design.

1.3.3 Seismic Ground Motion Considerations

The methodology for development of seismic ground motion criteria for design of both Aerial and Underground Structures (reflecting both the ODE and MDE) is documented in Chapter 2. The criteria should be developed on a site specific basis and based on 2009 probabilistic seismic hazard analysis procedures documented by the USGS and Caltrans. The procedures incorporate the latest consensus on active fault magnitude and recurrence relationships in the Los Angeles region, and on recently developed ground motion attenuation relationships. Any departure from these procedures due to new developments must be approved by Metro. Design considerations related to fault displacement estimates are also addressed in Chapter 2.

1.4 The LRFD Philosophy

The Federal Highway Administration (FHWA) and the Nation's states have established a goal that LRFD standards be incorporated in all new designs after 2007. In addition, most non-highway codes and standards have already or are beginning to follow suit in a trend that is extremely unlikely to be reversed. The Seismic Design Criteria documented in Chapter 3 have adopted the LRFD Philosophy.

Working stress design (WSD) began to be adjusted in the early 1970s to reflect the variable predictability of certain load types, such as wind loads, through adjusting design factors. This design philosophy is referred to as load factor design (LFD). A further philosophical extension results from considering the variability in the properties of structural elements, in similar fashion to load variabilities. While considered to a limited extent in LFD, the design philosophy of load-and-resistance factor design (LRFD) takes variability in the behavior of structural elements into account in an explicit manner. LRFD relies on extensive use of statistical methods, but sets forth the results in a manner readily usable by bridge and aerial guideway designers and analysts.

Applying the concepts of LRFD leads to an AASHTO specified design life of 75 years. Design Life as used here means the period of time on which the statistical derivation of transient loads is based. With the additional seismic and other precautions taken with the aerial structures, and the mainly static forces applied to underground structures, the service life for structures carrying rail transit as designed under these criteria is 100 years.

LRFD employs specified Limit States to achieve the objectives of constructability, safety, and serviceability. A Limit State is defined as a condition beyond which a structure or structural component ceases to satisfy the provisions for which it was designed. The resistance of components and connections are determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis.

This inconsistency is common to most current specifications as a result of incomplete knowledge of inelastic structural action.

LRFD uses extreme event limit states to ensure the structural survival of structures during a major earthquake or flood, or when there is a potential collision by rail or rubber tired vehicles. Extreme Event Limit States are considered to be unique occurrences whose return period may be significantly greater that the design life of the bridge.

LRFD also classifies structures on the basis of operational importance. Such classification is based on the social-survival-and/or security-defense requirements. Metro is responsible for declaring a structure or structural component to be operationally important.

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5. California Building Standards Commission, “California Building Code, California Code of Regulations, Title 24,” based on the 2006 International Building Code, 2007.

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CHAPTER 2

SEISMIC DESIGN GROUND MOTION CRITERIA

2.1 General

This Chapter describes the current Metro Seismic Design Ground Motion Criteria to be used for Aerial Guideways and Structures Chapter 3, Part A and for Underground Guideways and Structures Chapter 3, Part B. The Ground Motion Criteria replaces previous criteria used by Metro for projects as described in:

1. The “Supplemental Criteria for Seismic Design of Underground Structures” published in June 1984 by Metro Rail Transit Consultants.
2. The Section 5 Structural Supplement A “Ground Motion Response Spectra for Bridge and Elevated Structures Proposed Metro Rail Projects, Los Angeles County, California” prepared by Law/Crandall Inc, in 1994.
3. The 1997 Ground Motion Criteria developed for four stations for the Eastside Extension, prepared by Woodward Clyde Consultants.
4. The Structural/Geotechnical ground motion criteria prepared for Aerial Underground Structures (Appendices A and B) for the Eastside Extension design-build specifications.

Section 2.2 provides an overview of the Geologic and Seismic Environment related to existing or proposed Metro transportation alignments including descriptions of the regional stratigraphy, tectonics, historical seismicity, and principal active faults.

Section 2.3 describes the use of probabilistic seismic hazard analyses for the development of the site specific Operating Design Earthquake (ODE) and Maximum Design Earthquake (MDE) ground motion criteria for aerial and underground structures. Criteria development includes:

1. Determination of ground surface design spectra, and peak ground motion parameters for aerial structures.
2. Determination of design spectra at depths below underground structures for development of matching acceleration time histories.
3. Procedures for determining spectral matched acceleration time histories for analyses.

Section 2.4 discusses the evaluation of fault rupture potential and methods used to determine fault rupture characteristics and displacement estimates. Probabilistic methods for estimating fault displacements are also noted.

2.2 Geologic and Seismic Environment

2.2.1 Regional Stratigraphy

The existing and proposed Metro Transportation alignments traverse portions of four major physiographic features as shown in Figure 2-1, namely the Los Angeles Basin, the Santa Monica Mountains, the San Gabriel Valley, and the San Fernando Valley. The Los Angeles Basin, once a marine embayment, accumulated sediments eroded from surrounding highlands during the Miocene and Pleistocene epochs beginning about 25 million and one million years ago, respectively. Uplift of the Santa Monica Mountains provided much of the sediment filling the Basin. Volcanic activity also produced extensive accumulations of basalt in the Santa Monica Mountains during the Miocene epoch. The Los Angeles Basin and the San Fernando Valley were uplifted during the Pleistocene epoch. Rapid uplift and erosion was in early Pleistocene time, filling the Los Angeles Basin with about 1,300 feet of sandy sediments (San Pedro Formation). Holocene time (beginning with the last melting of the Ice Sheets 11,000 years ago) resulted in alluvium (coarse gravels and sands) being deposited in stream channels extending into the Los Angeles Basin. The San Fernando Valley has been filled with considerably thicker deposits of alluvial sediments than the northern part of the Los Angeles basin.

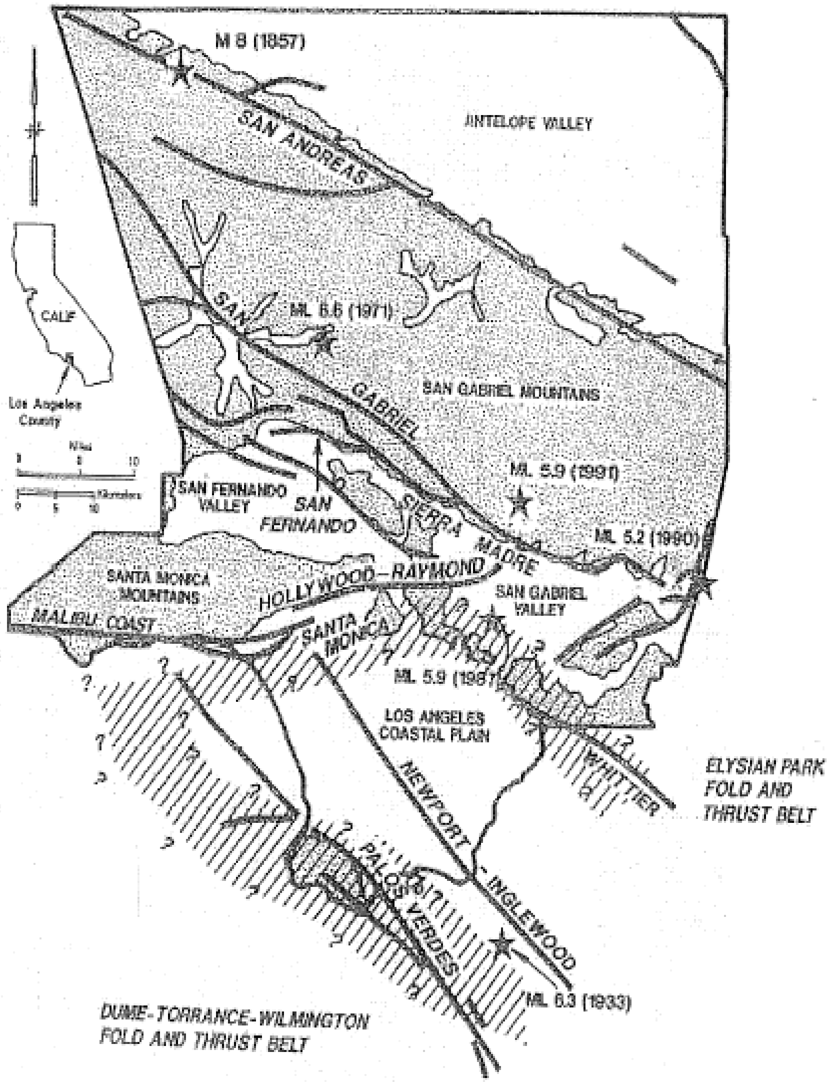


Figure 2-1 Location map of Los Angeles County showing physiographic provinces, selected faults and significant historic earthquakes. Fold and thrust belts shown from Hauksson (1990) represent potentially significant "blind" seismogenic sources. (After Gath, 1992)

Geologic units associated with existing or proposed tunnel alignments in order of increasing age, are shown in Table 2-1. With reference to this table, the geologic materials ranging from Alluvium through the Puente Formation can be regarded as being associated with soft ground or soft rock tunneling methods. The harder rock formations associated with the Topanga Formations and the granitic rocks encountered in the Santa Monica Mountains, require hard rock tunneling techniques.

Table 2-1 – Geologic Units Associated with Existing or Proposed Tunnel Alignments (after Converse et al. 1981)

Formation	Map Symbol	Description
Young Alluvium	(Qal)	Silt, sand, gravel, and boulders; chiefly unconsolidated (loose) and granular.
Old Alluvium	(Qalo)	Clay, silt, sand, and gravel; chiefly consolidated (stiff) and fine-grained.
San Pedro Formation	(Sp)	Sand; clean, relatively cohesionless; locally impregnated with oil or tar (Formation not exposed at surface on Geologic map).
Fernando Formation	(Tf)	Claystone, siltstone, sandstone; chiefly soft, stratified siltstone; local hard sandstone beds.
Puente Formation	(Tp)	Claystone, siltstone, sandstone; chiefly soft, stratified siltstone; local hard sandstone.
Topanga Formation	(Tt)	Siltstone, sandstone, conglomerate; chiefly hard, well cemented, massive sandstone; local soft, thin siltstone beds; includes some Cretaceous conglomerate and sandstone, undifferentiated beds.
Topanga Formation	(Tb)	Basalt; includes dolerite and andesitic basalt; non-columnar flows and intrusives; deeply weathered, soft crumbly at surface; hard, unweathered at depth.
Alluvial Fan	(Qf)	Silt, sand, gravel, and boulders; primarily semi-unconsolidated (dense) and granular.
Modelo Formation	(Tm)	Claystone, siltstone, sandstone; chiefly soft, diatomaceous stratified siltsone; local hard sandstone beds.
Granite	(Cg)	Chiefly granodiorites; deeply weathered, soft at surface; hard unweathered at depth.

The floor of the Basins are underlain by Quaternary-age sandy sediments with local silts, clays, and gravels. These generally can be subdivided into non-indurated loose Holocene-age sediments, and non-indurated, but denser, Pleistocene-age materials.

The uppermost Pleistocene materials are generally non marine deposits referred to as the Lakewood Formation which is on the order of 125,000 to 500,000 years old (California Department of Water Resources, 1961). These late- to middle-Pleistocene sediments overlie older, early-Pleistocene, marine sediments referred to as the San Pedro Formation which is more than 500,000 years old. The San Pedro Formation overlies marine Tertiary-age (> 2 million years) sediments and sedimentary rocks. These include the Pico, Repetto, Fernando, Puente, and Monterey formations. The Tertiary-age sediments and rocks, in turn, overlie Mesozoic-age (~100 million years) crystalline basement rocks at depths ranging from about 1,500 to 3,000 m west of the Newport-Inglewood Structural Zone (NISZ) to as much as 10,000 m in the deepest part of the central basin east of the NISZ (Yerkes et al., 1965). The basement west of the NISZ is primarily metamorphic rock (schist) whereas the basement to the east includes both metamorphic and igneous rocks.

2.2.2 Regional Tectonics

Except for the Newport-Inglewood Structural Zone, most surface geological faults such as the Santa Monica, Hollywood, and Whittier faults occur along the Basin margins. In addition to these known surface faults, the Los Angeles region is underlain by subsurface thrust and reverse faults (commonly referred to as "blind" faults and shown approximately on Figure 2-1 as dashed lines). These are poorly understood features with poorly known locations and orientations. Most of the known subsurface faults underlie the higher-standing plains along the inland margin of the Basin, but others have been proposed (for example, the San Joaquin Hills thrust fault). Most large earthquakes associated with these subsurface features are most likely to originate at depths between 10 and 15 km. The 1987 Whittier earthquake occurred on one of these buried faults that dips northerly under the Repetto Hills and San Gabriel Basin.

The present tectonic regime appears to have been in place since middle Pleistocene time and the present-day configuration of the Los Angeles Basin would have been recognizable about 200,000 to 300,000 years ago. The greatest tectonic activity within late Pleistocene time has occurred primarily in proximity to the major surface faults such as the Palos Verdes, Malibu-Santa Monica-Hollywood, Newport-Inglewood, Whittier, and Sierra Madre faults. The subsurface thrust faults within the region have not been active enough to create similar prominent uplifts and only a few (e.g. Santa Fe Springs) even have subtle recognizable surface expression.

2.2.3 Regional Seismicity

The southern California area is seismically active as shown on the seismicity map of Figure 2-2. Additional seismicity information is provided in Figure 2-3, which shows some of the more notable earthquakes in the Los Angeles Basin. Seismicity in the Los Angeles Basin does not clearly correlate to surface faults. There is no concentration or clustering of earthquakes in the site region except perhaps along the NISZ where a series of aftershocks from the 1933 event are located. Ward (1994) suggested that as much as 40% of the tectonic strain in southern California is not released on known faults. Part of this difficulty is due to the fact that the Basin is underlain by the several poorly known blind thrust faults as noted above.

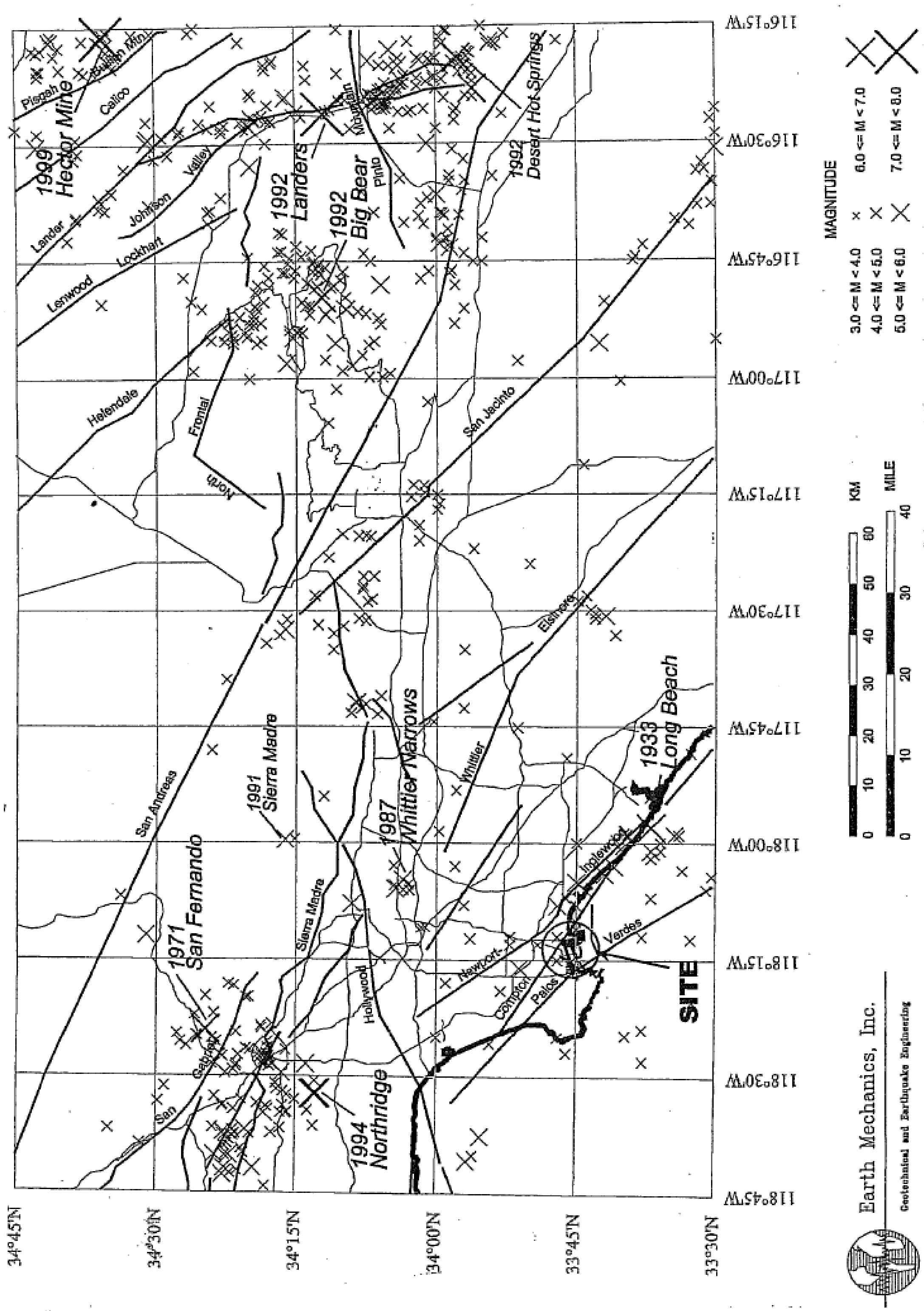
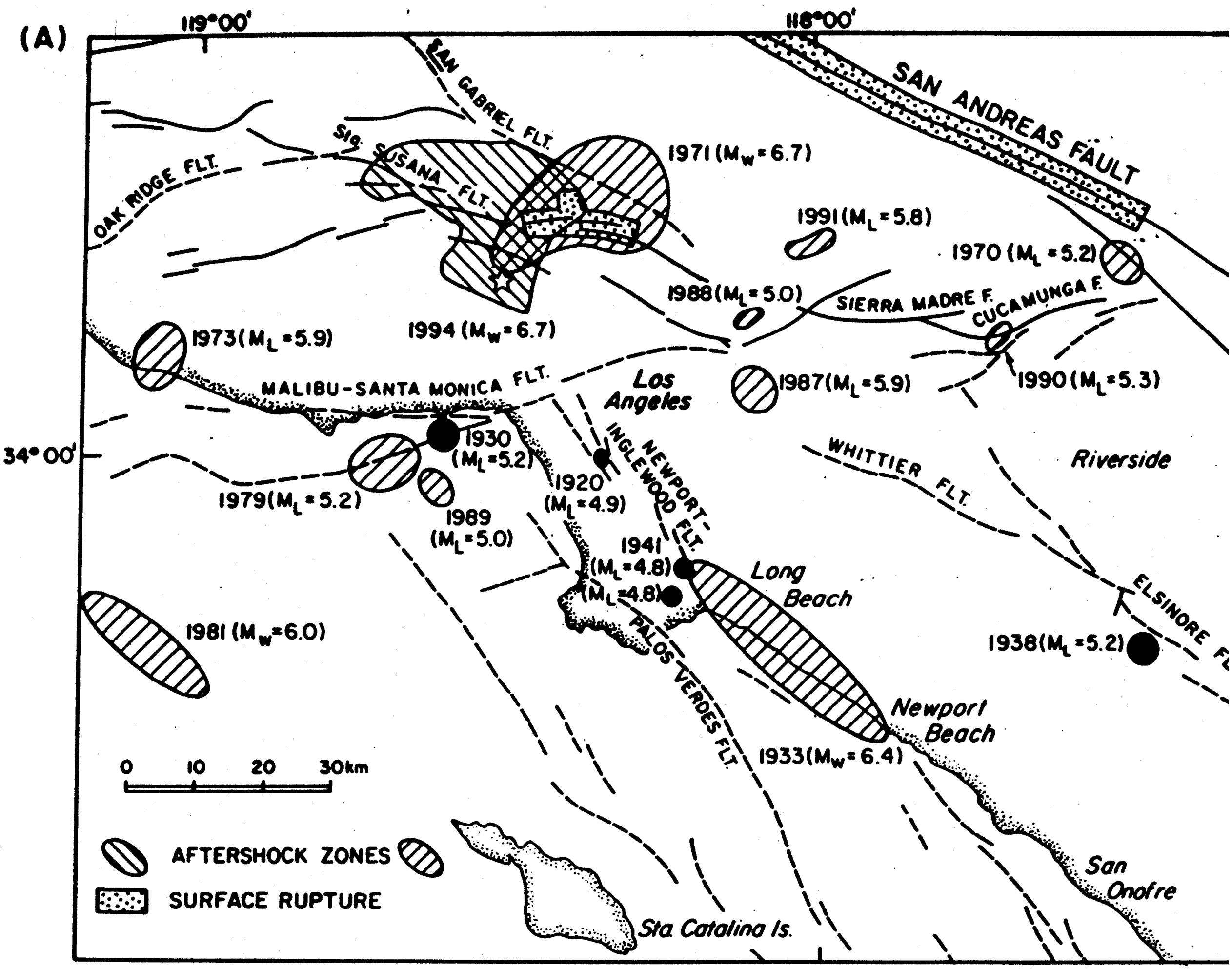


Figure 2-2 Seismicity Map



Note: Cross-hatched areas indicate aftershock zones (after Hauksson, 1995)

Figure 2-3 Significant Earthquakes in the Los Angeles Area

The largest historical earthquake within the Los Angeles Basin was the 1933 Long Beach earthquake of M_W = 6.4 (M_L = 6.3). The 1971 San Fernando (M_L = 6.4, M_W = 6.7) earthquake occurred outside of the basin along the northern margin of the San Fernando Valley within a zone of mapped surface faults of the Sierra Madre fault zone. The more-recent 1987 Whittier earthquake (M_L = 5.9, M_W = 5.9) and the 1994 Northridge (M_L = 6.4, M_W = 6.7) earthquakes occurred under the San Gabriel Valley and the San Fernando Valley, respectively, but were not associated with surface faults. In the offshore region, there have been no major earthquakes (M~7.0+) in historical times.

The 1933 Long Beach earthquake is generally believed to have been associated with the Newport-Inglewood Structural Zone (Benioff, 1938). This association was based on abundant ground failures along the trend but no unequivocal surface rupture was identified. Hauksson and Gross (1991) reevaluated the seismicity and relocated the 1933 earthquake to a depth of about 10 km below the Huntington Beach-Newport Beach city boundary.

Hauksson (1987, 1990) ~~analyzed~~ **analysed** the historical seismicity of the Los Angeles Basin. Although several older events were included, the principal time frame of the earthquake record studied was from 1977 to 1989, only about 12 years. This is a short time relative to the geologic time scales that control crustal tectonic activity, and thus the results of the study must be used cautiously. Also, there were few moderate and no large events in this record. History has shown repeatedly that small earthquakes are not necessarily indicative of where larger events will occur and/or of the nature of the principal tectonic regime. Of 244 earthquake focal mechanisms, 59% were predominantly strike-slip, 32% were reverse, and the rest were normal-fault mechanisms. All of the events were widely distributed and intermixed, and patterns are ambiguous. A large proportion of the strike-slip events occurred along the NISZ but the distribution is generally loosely scattered. More of the reverse mechanisms occurred north of the latitude of Palos Verdes Hills than to the south but like the strike-slip events the pattern is loose and typified by widely scattered events. Most of the normal-fault mechanisms occurred in the offshore area, but several also occur along the NISZ.

In overview, both the earthquakes and the geologic structures in the Los Angeles Basin appear to characterize tectonic environments whereby the northernmost part of the Basin, adjacent to and including the Santa Monica Mountains, is primarily a contractional tectonic regime (thrust and reverse faulting); the middle part of the Basin (to about a line connecting the north side of the Palos Verdes Hills-Signal Hill-Peralta Hills is a mixture of contractional and transcurrent (transpressional) structures, and the southern part of the Basin is primarily a transcurrent regime (strike-slip faulting).

Without a history of repeated large earthquakes within the basin, it is difficult to characterize the maximum earthquake potential. Neither the 1971 San Fernando, the 1987 Whittier, nor the 1994 Northridge earthquakes occurred within the Los Angeles Basin. However, they occurred within the same basic compressional tectonic regime and thus are probably representative of the size of earthquakes likely to occur on the larger subsurface reverse faults within the basin. The maximum earthquakes for the strike-slip faults can be estimated only from comparison of empirical fault-length/earthquake-magnitude data, and these suggest events in the M = 7 to 7.25 range.

2.2.4 Principal Active Faults

The following paragraphs briefly describe the principal active faults in the Los Angeles region that potentially could impact Metro structures. Locations of these faults are shown on Figure 2-1. This information is given from a regional perspective for understanding the nature of the faults, and provides a basis for the parameters used in probabilistic seismic hazard analysis discussed in Paragraph 2-3. More detailed descriptions of active faults in the Los Angeles Region may be found in publications by Schell and Dolan et al.

Palos Verdes Fault

The Palos Verdes fault extends from the northeast side of the Palos Verdes Peninsula southeasterly into deep water of the Continental Borderland. Northwesterly, the fault extends into Santa Monica Bay. Together, these segments extend for a total length of about 100 km.

The Palos Verdes fault is predominantly a strike-slip fault but has a small vertical component (~10% to 15%). The slip rate of the Palos Verdes fault is based primarily on the geophysical and geological studies in the outer harbor of the Port of Los Angeles by McNeilan et al. (1996). McNeilan et al. estimated a long-term horizontal slip rate of between 2.0 and 3.5 mm/yr. A slip rate of 3.0 mm/yr (±1mm) is the rate used by the California Geological Survey and the U.S. Geological Survey.

There have been no significant earthquakes on the fault since arrival of the Franciscan missionaries in the 1700s so there are virtually no direct data to help constrain the recurrence interval for large earthquakes on the Palos Verdes fault. Using the empirical data of Wells and Coppersmith (1994) to indirectly make judgments on how long it would take to store up enough strain to generate a M6.8 to 7.4 earthquake, it appears that recurrence intervals for such earthquakes on the Palos Verdes fault would range from a few hundred to a few thousand years. For example, fault rupture scenarios evaluated by McNeilan et al. (1996) ranged from 180 to 630 years for a M6.8 event, 400 to 440 years for a M7.1 event, 1,000-1,100 years for a M7.2 event, and 830 to 1,820 years for a M7.4 event).

Newport-Inglewood Structural Zone

The Newport-Inglewood Structural Zone (NISZ) consists of the northwest-southeast trending series of faults and folds associated with an alignment of hills in the western Los Angeles Basin extending from the Baldwin Hills on the north to Newport Mesa on the south (Figure 2-1). The fault seems to have originated in late Miocene time but based on relative stratigraphic thickness of bedding across the zone, the greatest activity seems to have occurred since Pliocene time indicating the fault is quite young.

The NISZ comprises several individual faults and branch faults, few of which have good surface expression as actual fault scarps.

The maximum earthquake used for the NISZ in local geotechnical investigations has generally been magnitude 7.0. This may be relatively small for a feature as long as the SMB zone but the magnitude is based on the concept that the zone consists of shorter discontinuous faults, or segments, that behave independently. The fault was the source of the 1993 Long Beach earthquake of magnitude 6.3, but as with the Palos Verdes fault, the history of earthquakes on the NISZ is incomplete so it is difficult to estimate a maximum earthquake. Empirical fault-length/ earthquake-magnitude relations (Wells and Coppersmith, 1994) suggest an MCE of about 7.0.

The recurrence interval for the maximum earthquake on the NISZ is very long, on the order of a thousand years or more (Schell, 1991; Freeman et al., 1992; Shlemon et al., 1995; Grant et al., 1997). The rate of fault slip is poorly known but seems to be very slow.

Sierra Madre Fault

The Sierra Madre fault is one of the major faults in the Los Angeles region and lies along the southern margin of the San Gabriel Mountains forming one of the most impressive geomorphic features in the Los Angeles area. The fault is recognized by juxtaposition of rock types, shearing and crushing along the fault trace, and by linear land forms (geomorphology). The fault is primarily a thrust fault that has thrust the ancient igneous and metamorphic rocks of the San Gabriel Mountains up and over young Quaternary-age alluvial deposits. The fault zone is very complex and over much of its length comprises several subparallel branches along the northern edge of the San Fernando and San Gabriel valleys (Figure 2-1). The fault may also be divided into segments along length, each with somewhat different rupture characteristics and histories.

The poor documentation of Quaternary faulting on the Sierra Madre fault makes it difficult to assess its earthquake capability. Based on worldwide empirical fault-length/earthquake-magnitude relationships (Wells and Coppersmith, 1994), the Sierra Madre fault is capable of producing earthquakes in the 7.0 to 7.5 magnitude range (Dolan et al., 1995). If the fault ruptures one of the segments independently, earthquakes of M = 7.0 are more likely; if more than one segment ruptures together, larger earthquakes are possible.

About 20 km of the westernmost part of the Sierra Madre fault ruptured the ground surface during the 1971 San Fernando earthquake (M_w = 6.7). The 1971 event was characterized by reverse faulting along a fault dipping about 45^o to 50^o northerly. In 1991, a magnitude 5.8 earthquake occurred below the San Gabriel Mountains at a depth of about 16 km and is generally believed to have occurred on the Clamshell-Sawpit branch of the Sierra Madre fault zone. The best available information indicates that large earthquakes on the Sierra Madre fault occur sometime between a few hundred years to a few thousand years (~5,000 years according to Crook et al, 1987). Geological and paleoseismological studies by Rubin et al. (1998) suggest that two prehistoric ruptures within the past 15,000 years had large displacements typical of earthquakes in the M = 7.0 to 7.5 range.

Reliable geological information on the slip rate of the Sierra Madre fault is scarce and the average time between large ground rupturing earthquakes is poorly known. Some geological studies have indicated that the average rate of displacement for the Sierra Madre fault may be as high as about 3 to 4 mm/year. The California Geological Survey uses a slip rate of 2.0 mm/yr (±1.0 mm).

Malibu Coast, Santa Monica, Hollywood Fault System (Southern Frontal Fault system)

One of the major fault systems in the Los Angeles Basin is along the southern edge of the Santa Monica Mountains separating Mesozoic plutonic rocks from Tertiary and Quaternary sedimentary rocks. The fault system consists of the Santa Monica and Hollywood faults and smaller segments such as the Malibu Coast and Potrero faults (see Figure 2-1). Together, these faults form the southern boundary fault of the Santa Monica Mountains.

The Santa Monica Mountains rise abruptly to 500 to 600 m above the Los Angeles Basin floor and are indicative of a large vertical component of faulting. Earthquake focal mechanisms and local geologic relationships suggest reverse faulting with a subordinate left-lateral component. Investigations is the past decade or so (e.g. Davis et al., 1989; Dolan et al., 1995) postulate that the Santa Monica and Hollywood fault are predominantly strike-slip features and that the mountains are underlain by a separate, but related, blind thrust fault. The Metro Rail Red Line tunnel through the Hollywood segment of the fault system revealed a major shear zone with the plutonic rocks of the Santa Monica Mountains, uplifted over Quaternary alluvium and colluvium. The fault zone consists of a northerly dipping fault with about a 100-meter-wide sheared gouge zone.

There have been no large earthquakes associated with Western Transverse Ranges southern boundary fault zone in historical time, but geological studies (Dolan et al., 1997, 2000a, 2000b) have documented Holocene faulting within the zone. Geological data indicate the recurrence intervals for large earthquakes are very long and appear to be on the order of a few thousand years; The Hollywood fault appears to have had one surface rupturing event in Holocene time (Dolan et al., 1997; 2000) with an average recurrence interval in the range of about 4,000 to 6,000 or 7,000 years. The Santa Monica fault has had two or probably three events in the past 16,000-17,000 years suggesting an average recurrence interval of about 7,000-8,000 years (Dolan et al., 2000).

Documented slip rates are less than 1.0 mm/yr but this estimate suffers from lack of data on the lateral slip. The California Geological Survey (2003) assumes a slip rate up to about 1.0 mm/yr (± 0.5 mm). The great length of the fault system suggests that it is capable of generating a large earthquake (M~7.5) but the discontinuous nature of faulting suggests that faults may behave independently and perhaps a smaller maximum earthquake (M~6.5 to 7.0) is more appropriate. Dolan et al (1997) postulated a M_w = 6.6 event for the Hollywood fault, and Dolan et al. (2000) postulated an M = 6.9-7.0 event for the Santa Monica fault.

Raymond Hill Fault

The Raymond Hill fault or as commonly referred to, the Raymond fault. The Raymond Hill fault is about 26 km long and extends approximately east-west through the communities of San Marino, Arcadia, and South Pasadena (Figure 2-1).

The Raymond Hill fault is characterized by left-lateral oblique reverse slip. This fault dips at about 75 degrees to the north. The rate of slip is between 0.10 and 0.22 mm/yr. The fault has been considered by some geoscientists to be interconnected with the Hollywood fault because they have similar trends and similar types of displacement. However, the disparity between recurrence intervals and the age of latest surface rupture suggests they are discrete features.

The most recent major rupture occurred in Holocene time, about 1000 to 2000 years ago (Weaver and Dolan, 2000). There is geological evidence of at least eight surface-rupturing events along this fault in the last 40,000 +/- years. At least five surface ruptures occurred in the past 40,000 years. However, four of these events occurred between 31,500 and 41,500 years ago (Weaver and Dolan, 2000). This indicates that surface ruptures occur over very irregular intervals and may be more random than systematic.

Elysian Park Fold and Thrust Belt

The Elysian Park Fold and Thrust Belt (EPFT) was initially a concept by Davis et al (1989) who postulated that the Los Angeles area is underlain by a deep master detachment fault, and that most of the folds and faults in the region result from slip along the detachment causing folding and blind thrust faulting at bends and kinks in the detachment fault. Shaw and Suppe (1996) further developed and expanded the detachment/blind thrust model. They proposed several zones of subsurface faulting and folding consisting of the Elysian Park trend, the Compton-Los Alamitos trend, and the Torrance-Wilmington trend. Few of these thrust ramps have actually been seen in well data or seismic-reflection surveys because the postulated features are generally at depths beyond the reach of drilling or seismic-reflection methods. The detachment/blind thrust model was initially embraced primarily because the 1987 Whittier Narrows earthquake occurred in proximity to one of the postulated thrust ramps beneath the Elysian Park fold belt. At present most seismic hazard analyses recognize only the Upper Elysian Park Thrust (see Figure 2-1).

Recurrence-interval estimates range from 340 to 1,000 years. Oskin et al. (2000) model the Upper Elysian Park thrust as extending from the Hollywood fault to the Alhambra Wash fault with a slip rate of 0.8 to 2.2 mm/yr and magnitude 6.2 to 6.7 earthquakes with recurrence intervals in the range of 500 to 1300 years. The California Geological Survey, following the lead of Oskin et al. (2000), models the Upper Elysian Park thrust as a feature about 18 km long and dipping 50^{o} northeasterly with a slip rate estimate of about 1.3 ±0.4 mm/yr.

Puente Hills Fault System

The Puente Hills Thrust fault system (PHT) is the name currently given to a series of northerly dipping subsurface thrust faults (blind thrusts) extending about 40-45 km along the eastern margin of the Los Angeles Basin. Shaw and Shearer (1999) synthesized oil-company geophysical data and seismicity to interpret three discrete thrust faults underlying the La Brea/Montebello Plain, Santa Fe Springs Plain, and Coyote Hills.

Down-dip projection of the Santa Fe Springs segment extends to the approximate depth of the 1987 Whittier Narrows earthquake which Shaw and Shearer (1999) relocated to about 15 km depth. The close association of seismicity to the fault projections indicates that the fault is seismically active. Shaw and Shearer proposed that the Puente Hills fault system is capable of generating about magnitude 6.5 to 7.0 earthquakes and has a slip rate of between 0.5 to 2.0 mm/yr.

Subsequent work on the fault system (Shaw et al., 2002) infers that the en echelon segments of the Puente Hills Thrust are related and displacements are gradually transferred from one segment to the next. Using empirical data on rupture area, magnitude, and coseismic displacement, Shaw et al. (2002) estimated earthquakes of M_W 6.5 to 6.6 for single segments and M_W 7.1 for a multi-segment rupture. The recurrence intervals for these events are on the order of 400 to 1,320 years for single events and 780-2600 years for magnitude 7.1 events.

Paleoseismological studies using trenching and borings at the surface projection of the Santa Fe Springs fault (Dolan et al., 2003) identified four buried folds. This deformation was interpreted to be a result of subsurface slip associated with M_W = 7.0± earthquakes within the past 11,000 years.

The most recent seismic hazard model by the California Geological Survey (2003) used a slip rate of 0.7 ± 0.4 mm/yr.

2.3 Probabilistic Seismic Ground Motion Criteria

2.3.1 Design Earthquakes – Probabilistic Seismic Hazard Spectra

As previously described, Metro earthquake design policy for both aerial and underground structures has been based on a two level probabilistic design approach since 1983, namely:

1. An operating design earthquake (ODE) defined as an earthquake event likely to occur only once in the design life, where structures are designed to respond without significant structural damage and

2. A maximum design earthquake (MDE) defined as an earthquake event with a low probability of occurring in the design life, where structures are designed to respond with repairable damage and to maintain life safety.

Current Metro design criteria assume a design life of 100 years. To establish probabilistic seismic ground motion criteria, design earthquake motions are defined as follows:

	Probability of Exceedance	**Return Period**
Operating Design Earthquake (ODE)	**50% in 100 years**	**144 years (say 150 years)**
Maximum Design Earthquake (MDE)	**4% in 100 years**	**2475 years (say 2500 years)**

Note that similar probabilistic criteria have been adopted by other rail transit agencies in the United States, including those in Seattle and New York.

The MDE and ODE levels of horizontal ground shaking are best developed based on probabilistic seismic hazard analyses due to the large degree of variation (or uncertainty) in the observed ground shaking in the strong motion database. A probabilistic approach can better take into account the uncertainty parameters in evaluating strong ground characteristics for design, including earthquake magnitude recurrence intervals for source zones and ground motion attenuation relationships. This philosophy is also consistent with the approach in other major projects for critical structures.

Probabilistic seismic hazard analyses to determine site specific design spectra require four major steps as shown in Figure 2.4. In step 1, Seismic Source Identification, the seismic sources capable of generating strong ground motions at the project site(s) are identified and the geometries of these sources (i.e. their location and spatial extent) are defined. In Step 2, Magnitude-Recurrence, a recurrence relationship describing the rate at which various magnitude earthquakes are expected to occur is assigned to each of the identified seismic sources. Step 1, Seismic Source Identification, and Step 2, Magnitude-Recurrence, together may be referred to as seismic source characterization. In Step 3, Ground Motion Attenuation, an attenuation relationship that describes the relationship between earthquake magnitude, site-to-source distance, and the ground motion parameter of interest is assigned to each seismic source for a specific ground stiffness condition (characterized by a shear wave velocity). In Step 4, Probability of Exceedance, the results from the first three steps are integrated to produce a curve relating the value of the ground motion parameter of interest at the site(s) of interest to the probability that it will be exceeded over a specified time interval.

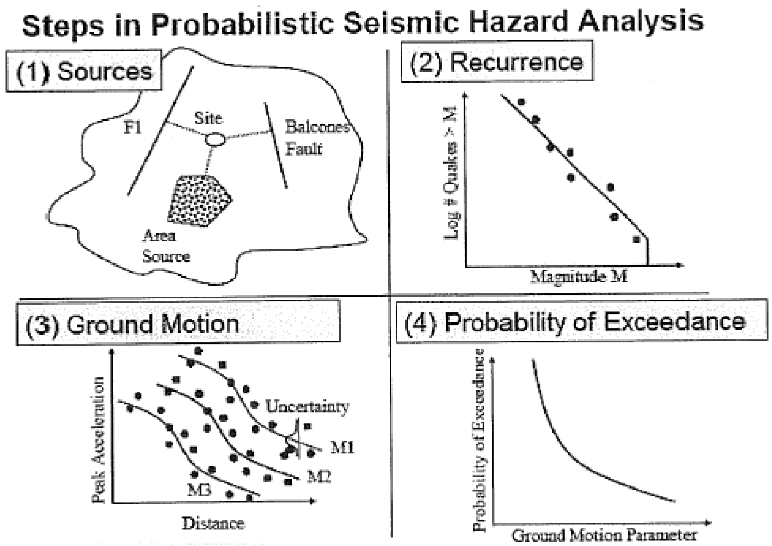


Figure 2-4 Steps in Probabilistic Seismic Hazard Analysis

The data used for Steps 1 through 3 are continually updated by seismologists and geologists as new research findings are debated and published. The probabilistic seismic ground motion criteria documented in previous Metro design guidelines used data which has been superceded by recent research. In particular, the most recent 2008 update of the USGS National Seismic Hazard Maps (USGS, 2008) incorporates the development of the Next Generation of Attenuation (NGA) relationships (Power et al. 2008) developed as a result of a 5 year research program coordinated by the Pacific Earthquake Engineering Research Program (PEER) in partnership with the USGS and the Southern California Earthquake Center (SCEC). In addition, updated fault parameters and fault source models were adopted from information developed by the Working Group on California Earthquake Probabilities (WGCEP) detailed in a 2008 WGCEP Report by Wells et al. (2008).

As a result of the above developments, Metro has adopted the 2008 USGS update of the National Probabilitic Seismic Hazard Analyses (PSHA) as a basis for their probabilistic ground motion criteria. In particular, ODE and MDE Spectral Accelerations for specific sites, should be developed using data available in the 2009 PSHA Interactive Deaggregation USGS Web Site as discussed below and illustrated in the example documented in Section 2.3.5.

2.3.2 Ground Motion Criteria- Aerial Structures

The seismic design of aerial and surface structures for Metro transportation projects, should be based on site specific ODE and MDE ground surface 5% damped acceleration spectra developed using the USGS 2009 PSHA Interactive Deaggregation Web Site (USGS, 2009). Input data requires the site coordinates and the average shear wave velocity in the top 30 meters (V_s^{30}) of the site. Spectral ordinates may be plotted for available period values of 0.0 (PGA), 0.1, 0.2, 0.3, 0.5, 1.0, 2.0, 3.0, 4.0 and 5.0 seconds. Note that the use of V_s^{30} replaces the use of site Soil Class Factors A-F used in older versions of USGS evaluations of site spectra. For the ODE, a 50% probability of exceedance in 100 years is available at the Web Site. However for the MDE, it is necessary to use a value of 2% in 50 years (equivalent to 4% in 100 years).

The determination of the design spectra requires values of V_s^{30} to be obtained at a site, where V_s^{30} is the average shear wave velocity for the upper 30 meters of the site profile. Insitu geophysical methods for determining V_s^{30} are documented in the Caltrans Geotechnical Services Design Manual (Caltrans 2009). Methods include using the Oyo Suspension P-S Logger (Nigbor and Imai, 1994), downhole shear wave measurements such as the Seismic CPT cone and Rayleigh Surface Wave Inversion Methods. Data from geophysical methods are required for final design. The use of empirical methods based on correlations of V_s^{30} with SPT blow counts, CPT data or undrained shear strengths of cohesive soils as described by Caltrans (2009), may be used for initial estimates in preliminary design, but only used for final design if supplemented by site specific calibration against geophysical data.
In addition to the MDE spectral evaluations described above, where Metro aerial structures impact a Caltrans "right-of-way", an additional evaluation of Caltrans design spectra is required, to check that the Caltrans spectral ordinates do not exceed the MDE values. In 2009 Caltrans adopted revised procedures for design spectrum development. The procedures use both a deterministic procedure based on Maximum Credible Earthquakes (MCE) on a revised California fault database and a probabilistic procedure using a 1000 year return period based on the 2008 update of the USGS Hazard Maps. For both procedures, spectral ordinate adjustments are made for near fault effects and deep basin effects. Input data requires the site coordinates and the average shear wave velocity in the top 30 meters (V_s^{30}) of the site. The online internet link for the web based spectra development procedure is **http://dap3.dot.ca.gov/shake_stable/**. The design spectrum is developed from the envelope of the deterministic and probabilistic spectral ordinates.

Note that the procedures for long period spectral ordinate corrections for deep basin effects documented by Caltrans should also be applied to the MDE USGS probabilistic spectra for Metro seismic design.

An example of both the Metro and Caltrans procedures for a specific site is given in Section 2.3.5.

2.3.3 Ground Motion Criteria- Underground Structures

The seismic design of underground structures for Metro transportation projects (including underground stations and tunnels) should be based on MDE subsurface ground motions expressed as site specific ground shear strains, velocities or displacements in the vicinity of the station walls or tunnels as required by the soil-structure interaction analyses discussed in the Supplemental Seismic Design Criteria (SSDC) for Underground Structures. The determination of the above ground motion parameters requires site specific one-dimensional non-linear site response analyses using computer programs such as SHAKE 91 (Idriss and Sun, 1992). The acceleration time histories required for such analyses should be determined from spectral matching procedures (described in paragraph 2.3.4 below) where the "rock outcrop" spectra is defined by the MDE ground acceleration from the USGS 2009 PSHA Website (USGS 2009), using a V_s^{30} value associated with a "rock outcrop" depth of at least 50 feet below the invert of the structure. (Note that the spectra should be adjusted for deep basin effects as previously discussed.) Appropriate strain dependent shear modulus and damping values for analyses should be assigned to site soil or rock strata at the site using accepted relationships such as those for cohesive soil (Vucetic and Dobry, 1991) and sands (EPRI, 1993), and where the maximum shear modulus is determined from measured shear wave velocities.

2.3.4 Spectral Matching of Acceleration Time Histories

As noted above, acceleration time histories are required for nonlinear analyses of aerial structures or for non-linear site response analyses for underground structures. The use of spectrum compatible input time histories has been widely used for major seismic design projects and is adopted for Metro projects. The concept ensures that a broad range of frequency content is included in the ground motion time history generated for design.

A reference ground motion time history (usually an actual earthquake record) is chosen as a “seed” or start-up motion and is gradually modified through an interactive process so that the response spectrum and the modified time history is compatible with the target spectrum. The recorded time histories should be chosen to match the site soil conditions and dominant earthquake magnitude and distance of the dominant earthquakes contributing to the design spectrum. The earthquake record database available through the Consortium of Organizations for Strong-Motion Observation Systems (COSMOS) is a valuable source of records. The web address is **http://db.cosmos-eq.org**.

Various methods have been developed to perform spectrum matching. A commonly used frequency domain method adjusts the Fourier amplitude spectrum based on the ratio of the target response spectrum to the time history response spectrum while keeping the Fourier phase of the reference history fixed. An alternative time domain approach for spectral matching adjusts the time history in the time domain by adding wavelets to the reference time history. A formal optimization procedure for this type of time domain spectral matching was first proposed by Kaul (1978) and was extended to simultaneous matching at multiple damping values by Lilhan and and Tseng (1987, 1988). Abrahamson (1998) also documents a procedure. While this approach is more complicated than the frequency domain method, it has good convergence properties.

There are relative merits for both the frequency and time domain approaches. However, the best approach would be that which makes the least changes to the startup motion. Figure 2-5 shows an example of spectral matching for using 1940 Imperial Valley earthquake recorded at El Centro, as a start-up motion.

Due to the variability in time history characteristics from “seed” motions, a minimum of three time histories should be used for nonlinear response analyses, and maximum response values of interest from the analyses used for design. A preferable approach is to use seven sets of time histories, and adopt the mean response values for design.

2.3.5 Example- Site Spectra Development

An example of the recommended approach to develop MDE and ODE Acceleration Response Spectra at a Bridge Site is shown in Figure 2-6. The site is the Metro Gold Line Bridge near Union Station, where it is assumed the value of V_s^{30} is 1000ft/sec. Note that the MDE spectrum is only slightly greater than the Caltrans deterministic spectrum for periods greater than 1.0 seconds. The adjustment factors for deep basin effects at the site were negligible. However, the Caltrans adjustments factors for near fault effects (The Puente Hills blind thrust fault) were significant for periods greater than 0.5 seconds. Near fault adjustment factors to probabilistic MDE spectra are not recommended for design at this time, due to uncertainties in the appropriate methodology for making such an adjustment to a probabilistic spectrum.

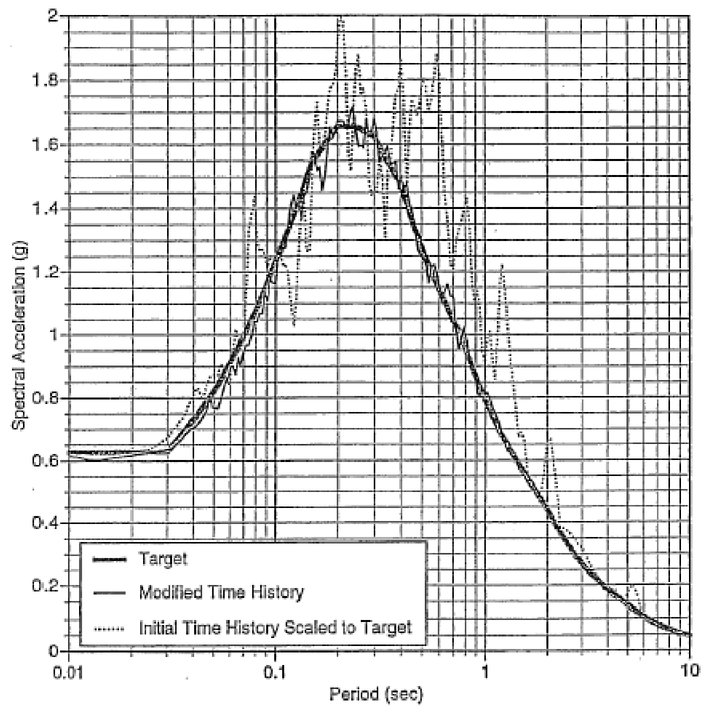


Figure 2-5 Example of Acceleration Spectral Matching

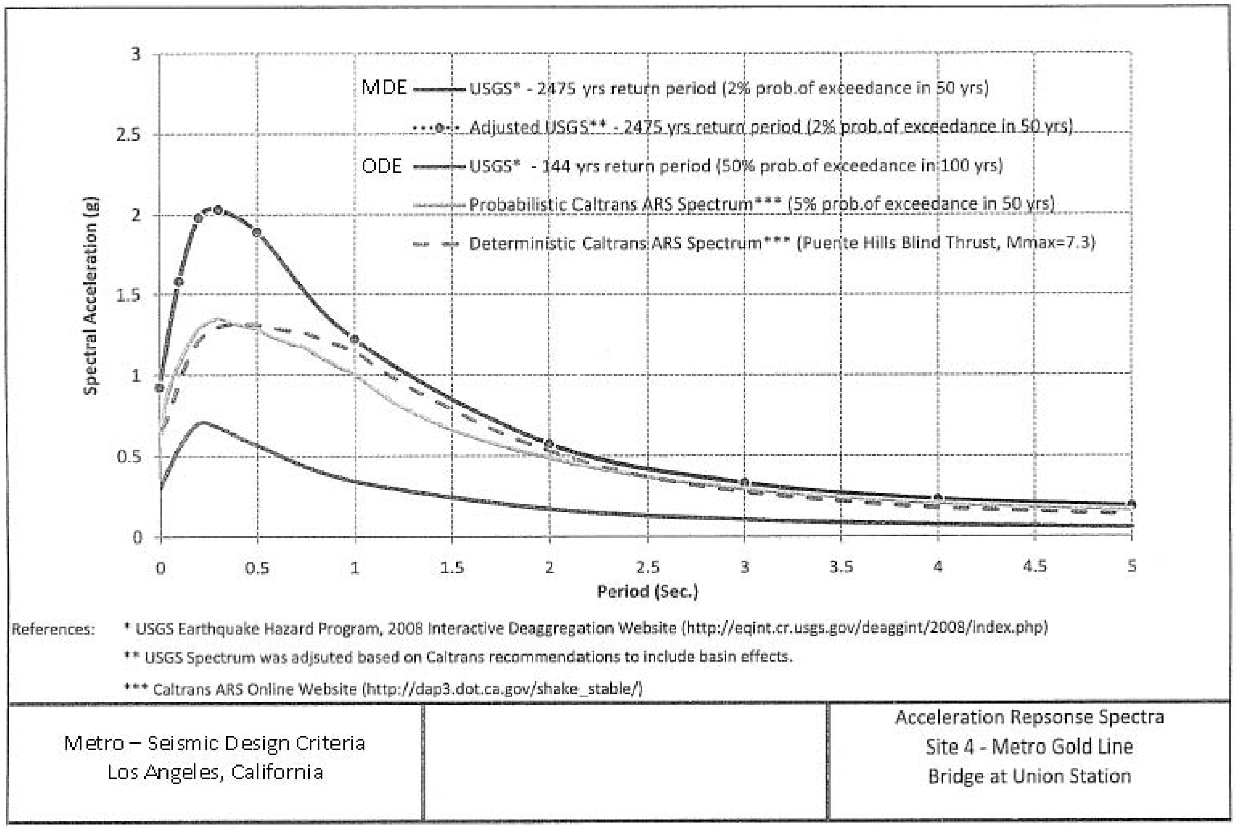


Figure 2-6 Example showing MDE and ODE Spectra at a Bridge Site

2.4 Surface Fault Rupture Displacement

2.4.1 Fault Rupture Potential Evaluation

Fault rupture refers to the shearing displacements that occur along an active fault trace when movement on the fault extends to the ground surface, or the depth of a tunnel or underground station. Displacements can range from inches to several feet. Because fault displacements tend to occur abruptly, often across a narrow zone, fault rupture can be very damaging to a bridge, or tunnel structure.

Fault ruptures generally are expected to occur along existing traces of active faults. Faults are generally considered to be active faults with a significant potential for future earthquakes and displacements if they have experienced displacements during the past approximately 11,000 years (Holocene time).

Design philosophy for fault crossings recognizes that it is difficult to prevent damage in a strong earthquake, given the magnitude of fault displacements. For tunnels, the general design philosophy now widely accepted for a fault crossing is to “overbore” the tunnel, so that if the maximum design earthquake-induced displacement occurs, the tunnel is still of sufficient diameter to fulfill its function after repairs. The overbored section is taken through the fault zone with transition zones narrowing to the regular tunnel diameter.

The overbored sections are backfilled with easily re-minable and crushable material such as “cellular” concrete. Such an approach was adopted for the North Outfall Replacement Sewer Project Tunnel when crossing the Newport Inglewood Fault, and was adopted for the Metro Red Line Segment 3 Hollywood Fault crossing. For blind thrust faults in the vicinity of underground structures, it may be necessary to estimate surface uplift, as in the case of the Eastside Coyote Escarpment (Habioma, et al., 2006).

Bridges crossing or immediately adjacent to active faults may be subjected to large differential displacements between adjacent piers and/or abutments due to surface faulting. A conservative design approach should be adopted if surface faulting is possible. Continuous spans are preferable. Adding extra confinement in the plastic hinge zones of the substructure might be used to provide the maximum displacement capacity. Simple spans can tolerate relative movements, but it will be difficult to ensure that the spans do not become unseated. To minimize this risk, very generous support should be provided.

The additional redundancy in continuous superstructures that are integral with their substructures will reduce the probability of total collapse. There is, however, a practical limit to the amount of relative displacement across a fault that can be accommodated in a monolithic structure. One alternative is to support a continuous superstructure on elastomeric bearings at each pier and abutment. These bearings can accommodate relatively large displacements and still provide an elastic restoring force to the superstructure. Restrainers may also be provided if gross movements are expected. Note that acceleration records from recent earthquakes indicate vertical accelerations in excess of 1.0 g in the near-field of the fault. In these situations, integral construction is preferred, but if elastomeric bearings are used, vertical restrainers should be provided to limit the uplift.

If an active fault exists in the vicinity of a project site detailed fault evaluations should be carried out by the geotechnical consultant oriented toward:

1. Establishing the location of the fault or fault zone relative to a project,
2. Establishing the activity of the fault if it traverses the project site, that is, the timing of the most recent slip activity on the fault, and
3. Evaluating fault rupture characteristics; i.e., amount of fault displacement, width of zone of displacement, and distribution of slip across the zone for horizontal and vertical components of displacement. For blind thrust faults, an assessment of surface uplift may be required. A probabilistic assessment of the likelihood of different magnitudes of fault displacement during the life of the structure may also be useful in decision-making.
4. The ground rupture characteristics for the design earthquake on the fault (e.g. type of faulting as illustrated in Figure 2-7), amount of slip and distribution into strike-slip and dip-slip components, and width of the zone of ground deformation.

A walk-down of the site and its vicinity should be conducted to observe unusual topographic conditions and evaluate any geologic relationships visible in cuts, channels, or other exposures.

Faults obscured by overburden soils, site grading, and/or structures can potentially be located by one or more techniques. Geophysical techniques such as seismic reflection or refraction surveying provide a remote means of identifying the location of steps in a buried bedrock surface and the juxtaposition of earth materials with different elastic properties. Geophysical surveys require specialized equipment and expertise, and their results may sometimes be difficult to interpret. Trenching investigations are commonly used to expose subsurface conditions to a depth of 15 to 20 feet. While expensive, trenches have the potential to locate faults precisely and provide exposures for assessing their slip geometry and slip history. Borings can also be used to assess the nature of subsurface materials and to identify discontinuities in material type or elevation that might indicate the presence of faults.

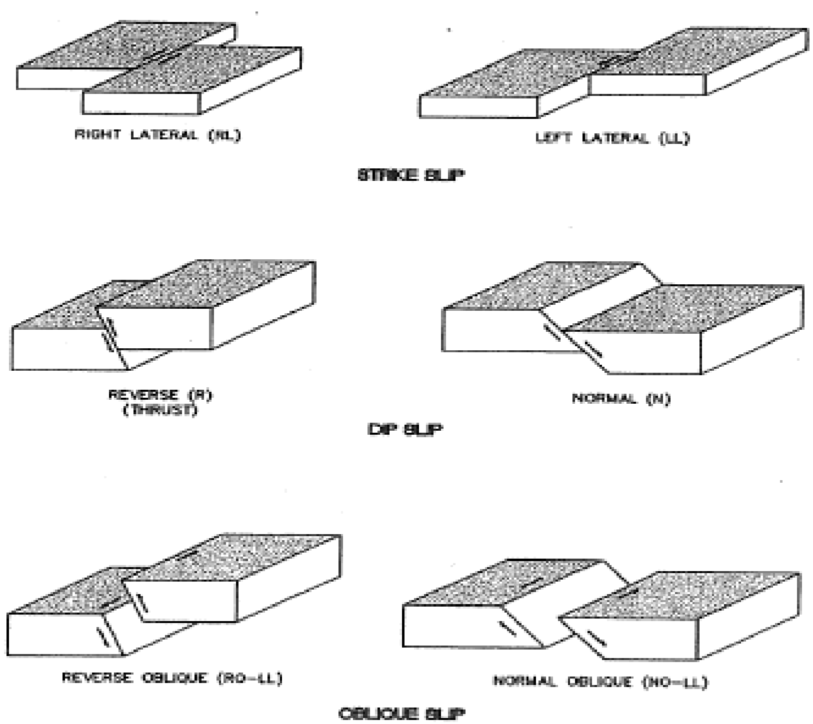


Figure 2-7 Types of earthquake faults

If it is determined that faults pass beneath the site, it is essential to assess their activity by determining the timing of the most recent slip(s) as discussed below. If it is determined, based on the procedures outlined below, that the faults are not active faults, further assessments are not required.

The most definitive assessment of the recent history of fault slip can be made in natural or artificial exposures of the fault where it is in contact with earth materials and/or surfaces of Quaternary age (last 1.8 million years). Deposits might include native soils, glacial sediments like till and loess, alluvium, colluvium, beach and dune sands, and other poorly consolidated surficial materials. Surface might include marine, lake, and stream terraces, and other erosional and depositional surface. A variety of age-dating techniques, including radiocarbon analysis and soil profile development, can be used to estimate the timing of the most recent fault slip.

2.4.2 Fault Rupture Characteristics and Displacement Estimates

Several methods can be used to estimate the size of future displacements. These include:

1. Observations of the amount of displacement during past surface-faulting earthquakes.

2. Empirical relations that relate displacement to earthquake magnitude or to fault rupture length.

The most reliable displacement assessments are based on past events. Observations of historical surface ruptures and geologic evidence of paleoseismic events provide the most useful indication of the location, nature, and size of the future events. Where the geologic conditions do not permit a direct assessment of the size of past fault ruptures, the amount of displacement must be estimated using indirect methods. Empirical relations between displacement and earthquake magnitude based on historical surface-faulting earthquakes (e.g. Wells and Coppersmith, 1994) provide a convenient means for assessing the amount of fault displacement. An example of such a relationship is shown in Figure 2-8. In this example, maximum displacement along the length of a fault rupture is correlated with earthquake magnitude. Maximum displacement typically occurs along a very limited section of the fault rupture length. Relationships are also available for the average displacement along the rupture length. Data from well-documented historical earthquakes indicate that the ratio of the average displacement to the maximum displacement ranges between 0.2 and 0.8 and averages 0.5 (Wells and Coppersmith, 1994). Other methods for calculating the average size of past displacements include dividing the cumulative displacement by the number of events that produced the displacement, and multiplying the geologic slip rate by the recurrence interval.

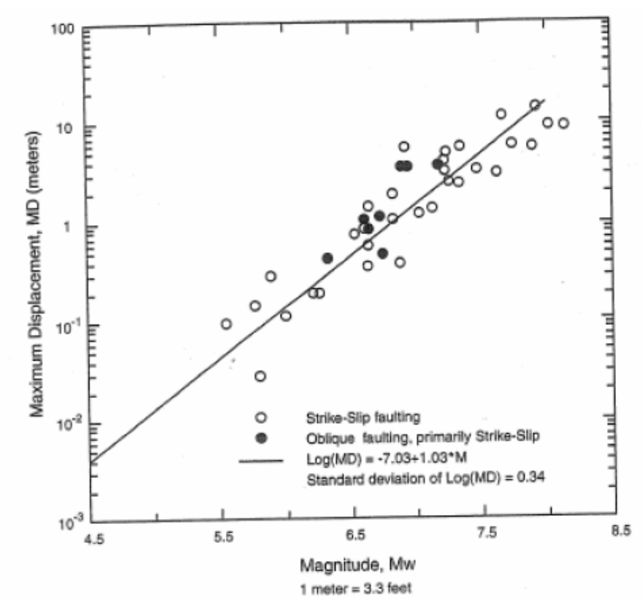


Figure 2-8
Relationship between maximum surface fault displacement and earthquake moment magnitude for strike-slip faulting (after Wells and Coppersmith, 1994)

Predicting the width of the zone and the distribution of slip across the zone of surface deformation associated with a surface faulting event is more difficult than predicting the total displacement. The best means for assessing the width of faulting is site-specific trenching that crosses the entire zone. Historical records indicate that the width of the zone of deformation is highly variable along the length of a fault. No empirical relationships having general applicability have been developed that relate the size of the earthquake or the amount of displacement on the primary fault trace to the width of the zone or to the amount of secondary deformation. The historical record indicates, and fault modeling shows that the width of the zone of deformation and the amount of secondary deformation tend to vary as a function of the dip of the fault and the sense of slip. Steeply dipping faults, such as vertical strike-slip faults, tend to have narrower zones of surface deformation than shallow-dipping faults. For dipping faults, the zone of deformation is generally much wider on the hanging wall side than on the foot wall side. Low-angle reverse faults (thrust faults) tend to have the widest zones of deformation.

2.4.3 Probabilistic Fault Displacement Evaluation

Probabilistic methods for assessing the hazard of fault rupture have been developed that are similar to the probabilistic seismic hazard analysis (PSHA) methods used to assess earthquake ground motions (Youngs et al., 2003). A PSHA for fault rupture defines the likelihood that various amounts of displacement will be exceeded at a site during a specified time period.

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CHAPTER 3

SUPPLEMENTARY SEISMIC DESIGN CRITERIA (SSDC)

Part A METRO SSDC FOR AERIAL GUIDEWAYS AND BRIDGES

3A1.0 SCOPE

This Seismic Design Criteria Revision updates the latest documents prepared in 2003 for Metro Gold Line Eastside Extension. This consisted of the Metro's Design Criteria Section 5 references to seismic design of structures and Section 5 Appendix Chapter 3 Part A for Aerial Guideways and Bridges and the addition of Section 5 Appendix Chapter 3 Part B for Underground Structures (Both referred to herein as Metro SSDC).

These design criteria apply to the design of normal aerial guideway, bridges, and structures to resist the effects of earthquake motions. Normal guideway and bridges should be considered to be new and conventional slab, beam, girder, and box girder superstructures with spans not exceeding 500 feet. These criteria do not apply to Critical/Essential Bridges as defined by the Caltrans BDS.

These criteria are intended to be used by designers who are experienced in the field of the bridge design and are familiar with the recent procedures being used by Caltrans.

Some special structures and structural systems involve unique design and construction problems not covered by this criteria, the provisions in this criteria govern only where applicable. Retrofit repairs, alterations and additions necessary for the preservation and restoration of historic buildings, bridges and structures may be made without strict conformance to this criteria when authorized by Metro and the governing agency. The use of any material or method of construction and design not specifically prescribed herein may be used upon approval by Metro and the governing agency.

All new structures shall be designed to resist the earthquake forces (EQ) and the ground displacement stipulated in these criteria. Aerial structures are defined as those elevated guideway structures which support Metro vehicles.

The requirement for peer review of the design work for Metro aerial structures and bridges will be determined by the Metro on a case by case basis.

3A2.0 DESIGN POLICY

The Metro Rail Project is a large-scale public project in an area highly susceptible to major earthquakes. Further, earthquake-initiated failures of selected structures and systems could lead to loss of life. For this reason Metro has developed special earthquake protection criteria for the project.

As previously discussed, the guiding philosophy of earthquake design for the project is to provide a high level of assurance that the overall system will continue operating during and after an Operating Design Earthquake (ODE). Operating procedures assume safe shut down and inspection before returning to operation. Damage, if any, is expected to be minimal and to minimize the risk of derailment of a train on the bridge at the time of the earthquake. Further, the system design will provide a high level of assurance that public safety will be maintained during and after a Maximum Design Earthquake (MDE). The definition of ODE and MDE levels as discussed in Chapter 2 is as follows:

The ODE is defined as the earthquake event which has a return period of 150 years. Such an event can reasonably be expected to occur during the 100-year facility design life. The probability of exceedance of this level of event is approximately 50 percent during the facility life.

The MDE is defined as the earthquake event which has a return period of 2500 years. Such an event has a small probability of exceedance during the facility life. This probability is approximately four percent or less.

3A3.0 PERFORMANCE OBJECTIVES

For the Operating Design Earthquake (ODE) which is likely to occur about once during the normal life expectancy, there shall be no interruption in rail service during or after the ODE. When subjected to the ODE, structures shall be designed to respond essentially in an elastic manner as defined by Caltrans Seismic Design Criteria, Section 3.2, Material Properties for Concrete Components, latest version. There shall be no collapse, and no damage to primary structural elements. Only minimal damage to secondary structural elements is permitted, and such damage shall be minor and easily repairable. The structure shall remain fully operational immediately after the earthquake, allowing a few hours for inspection.

For the Maximum Design Earthquake (MDE) which has a low probability of being exceeded during the normal life expectancy, some interruption in rail service is permitted to allow for inspection and repairs following the MDE. When subjected to the MDE, it is acceptable that the structures behave in an inelastic manner. There shall be no collapse and no catastrophic inundation with danger to life, and any structural damage shall be controlled and limited to elements that are easily accessible and can be readily repaired. The structure should be designed with adequate strength and ductility to survive loads and deformations imposed on the structure during the MDE, thereby preventing structure collapse and maintaining life safety.

In no case is the design to result in less seismic performance capability than that required by Caltrans BDS. To substantiate that this requirement has been met, a design check calculation using Caltrans criteria may be necessary.

3A4.0 CODES AND STANDARDS

These criteria make reference to, incorporate, are based on, or modify the following principal design codes:

1. For bridges and aerial structures that support rail transit loadings, except as otherwise noted herein, use the current Caltrans Seismic Design Criteria, latest edition, but with Metro specified rail transit loading. All the above is referred to throughout these criteria as "Caltrans SDC".

Caltrans Seismic Design Criteria (Current Version) shall supersede all provisions for seismic design, analysis, and detailing of bridge contained in the *AASHTO LRFD Bridge Design Specifications*. The Caltrans Seismic Design Criteria is used in conjunction with the Extreme Event I Load Combination specified in Caltrans BDS.

Where Caltrans SDC is not applicable, use the most appropriate code provided below.

2. For bridges that support railroad loadings, use the design requirements of the applicable railroad. In the absence of such requirements, use AREMA, Manual for Railway Engineering, Volume 2, Section 9, Seismic Design for Railway Structures, Latest Edition.

3. For bridges that support highway loading, use the design requirements of the applicable jurisdiction. In the absence of such requirements, use the Caltrans SDC.

4. For additional applicable codes, see the Structural/Geotechnical Criteria Section 5.1.2, Reference Data, and Section 5.1.3 Reference Codes.

3A5.0 DESIGN RESPONSE SPECTRA

Chapter 2 of this Metro SSDC describes the seismic design ground motion criteria to be used for bridges and aerial guideways. It provides an overview of the Geologic and Seismic Environment related to existing or proposed Metro transportation alignments including descriptions of the regional stratigraphy, tectonics, historical seismicity, and principal active faults. Chapter 2 also describes the use of probabilistic seismic hazard analyses for the development of the site specific Operating Design Earthquake (ODE) and Maximum Design Earthquake (MDE) ground motion criteria for bridges and aerial guideways.

The ground motion response spectra for this supplemental criteria are developed by the geotechnical consultant for each project site.

3A6.0 DESIGN GROUND MOTIONS

All aerial structures and bridges shall be designed to resist earthquake motions in accordance with Metro Seismic Design Ground Motion Criteria, Chapter 2. Where conflicts occur, the more critical will control. In some cases, aerial structures with bridges may be under other agency jurisdictions (such as Caltrans) and design criteria specified elsewhere. If seismic ground motion spectra are greater than those specified in Chapter 2, the former should be used for design.

Elements of above ground station structures not subject to rail transit loading shall be designed to resist earthquake motions in accordance with the applicable building codes of Section 5.1.

3A7.0 METHODS OF ANALYSIS

A complete aerial guideway or bridge system shall be composed of a single frame or a series of frames separated by expansion joints and/or articulated construction joints. A guideway or bridge shall be composed of a superstructure and a supporting substructure. Individual frame sections shall be supported on their respective substructures, consisting of piers, single column or multiple column bents that are supported on their respective foundations.

The determination of the seismic response of a bridge shall include the development of an analytical model followed by the response analysis of the analytical model to predict the resulting dynamic response for component design. Both the development of the analytical model and the selected analysis procedure shall be dependant on the seismic hazard (See Chapter 2), the selected seismic design strategy and the complexity of the guideway or bridge. The level of refinement in the analytical model and analytical procedure to be used shall be subject to the approval of Metro.

For additional information, see Caltrans SDC, latest version.

Due to the soil/structure interaction (SSI), three directional foundation soil springs shall be included in all of the analytical models. For in-structure displacement compatibility, column P-Δ effect shall also be included.

3A8.0 DESIGN FORCES, MOMENTS AND DISPLACEMENTS

Load Case 1: 100% of the absolute value of forces, moments, and displacements in transverse direction are added to 30% of the corresponding force and moments from the longitudinal and vertical directions.

Load Case 2: 100% of the absolute value of forces, moments, and displacements in the longitudinal direction are added to 30% of the corresponding forces and moments from the transverse and vertical directions.

Load Case 3: 100% of the absolute value of forces, moments, and displacements in vertical direction are added to 30% of the corresponding forces and moments from the transverse and longitudinal directions.

All aerial structures and bridges shall be designed to resist earthquake motions in accordance with Metro Supplemental Seismic Design Criteria (Metro SSDC) and as provided in Section A4.0. Where Metro SSDC and Caltrans SDC conflict, the more critical will control.

Use the Caltrans BDS method for the design of all structural components and connections. Each component and connection shall satisfy each of the following LRFD limit states, unless noted otherwise in another area of this criteria.

LRFD employs specified limit states to achieve the objectives of constructability, safety, and serviceability. See the Structural/Geotechnical Criteria Section 5.2.17. A Limit State is defined as a condition beyond which a structure or structural component ceases to satisfy the provisions for which it was designed. The resistance of components and connections are determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current specifications as a result of incomplete knowledge of inelastic structural action.

LRFD uses Service Limit states to provide for restrictions on stress, deformations, and crack width under regular service conditions. (See Structural/Geotechnical Criteria Section 5.2.17.1, Service limit state.)

LRFD uses extreme event limit states to ensure the structural survival of structures during major earthquakes. Extreme Event Limit States are considered to be unique occurrences whose return period may be significantly greater that the design life of the bridge. Extreme Event IA is the load combination relating to the operational use of the guideway that incorporates the ODE level seismic event. Extreme Event Limit State I is the load combination relating to the operational use of the guideway that incorporates the MDE level seismic event. (See Structural/Geotechnical Criteria Section 5.2.17.4, Extreme event limit state.)

For loading combinations, refer to Section 5.2.20 of the Structural/Geotechnical Criteria.

3A9.0 STRUCTURAL DESIGN

3A9.1 Properties for Material Components

For concrete and reinforcing steel, apply Caltrans SDC, Section 3.2 Material Properties for Concrete Components. In areas where this code is silent, the following shall apply.

Reinforcing steel shall be ASTM grade A706, with a yield strength between 66 ksi and 78ksi. Use a yield strength of 66 ksi unless restricted otherwise by the Building Code requirements.

The required performance for rebar development lengths is based on the Building Code's implemented ACI 318-08 and Caltrans BDS (See Structural/Geotechnical Criteria Section 5.1.3.C.1. The following statements provide additional requirement for use of #14 and #18 bars in seismic zones:

Straight #14s and #18s utilized in seismic zones shall be confined over the full length of the bar development zone. All other rebar development criteria and modification factors of the Building Codes referenced Chapter 12 of ACI 318-08, and Caltrans BDS implemented AASHTO LRFD Bridge Design Specifications, Section 5.11 shall be applied to straight bars.

When hooked #14s are utilized in seismic zones they shall be confined over the entire straight length of the bar development zone. No reduction factors may be applied to the basic development length of a standard hook in tension. In seismic zones #18 bars shall not be hooked. All other rebar development criteria and modification factors of Building Codes referenced Chapter 12 of ACI 318-08, and Caltrans BDS implemented AASHTO LRFD Bridge Design Specifications, Section 5.10 shall be applied to hooked bars.

3A9.2 Superstructure

The ultimate capacity of the superstructure in the area over the bent caps shall be designed for the larger of the forces resulting from ODE analysis or from the column plastic hinging.

The width of the superstructure assumed to be available to resist these forces shall be taken as one half of the column width plus the depth of the superstructure, on either side of the column centerline. For open soffit girders, follow Caltrans SDC 7.2.1.

When it is not possible to place the reinforcement within the above specified area, designers should then consider such details as:

Using thicker soffits and/or top slabs.
Widening the cap (but not more than d/2).
Using dropped cap.

Make all top and bottom bent cap main reinforcement continuous. If this is physically impossible, then at least 75% of reinforcement shall be made continuous. No lap splices shall be allowed in the main cap reinforcement.

3A9.3 Columns

Design column for essentially elastic behavior at the ODE level. At the MDE design check, make certain the plastic hinges occur at the top and/or bottom of the column. To transfer shear forces from plastic hinges, the joint shear and the additional longitudinal/transverse reinforcement shall be designed in accordance with section A9.4.

Column deflection capacity must be larger than displacement demand at MDE level. The following interaction equation must be satisfied:

$$\Delta_D < \Delta_C$$

Where:

Δ_D = Maximum displacement demand
Δ_C = Displacement capacity

Displacement demands amplification due to P-Δ effects must be considered.

Column reinforcement ratio should be kept under 4% to reduce congestion due to added joint reinforcement. It will also help in keeping the joint shear stresses lower than the maximum of $12(f'c)^{1/2}$.

For column flares design and detailing, refer to SDC, Section 7.6.5.

3A.9.4 Joint Shear

For joint shear, refer to Caltrans BDS implemented AASHTO LRFD Bridge Design Specifications, with California Amendments and AASHTO Guide Specifications for LRFD Seismic Bridge Design, latest editions.

The maximum shear stress in the enlarged joint area is to be limited to $12(f'c)^{1/2}$.

3A9.5 Outrigger Bents

In addition to the worst combination for a particular design case, the outrigger bents shall be designed for dead load forces increased and decreased by 50%.

For short outriggers (i.e. outrigger length less than the larger cap cross-section dimension), an additional check for torsional shear friction is to be performed.

Outrigger joints can be pinned at the top to reduce torsional moments in the cap beam.

3A9.6 Expansion Joints

Design expansion joints, calculate the movement rating "MR" and gap size "a" according to the following conditions.

$MR = 2\Delta EQL + \Delta T/2$ > MR required by BDS.

$a = \Delta EQL - \Delta PS/2 + \Delta T/4$

or,

$a = EQU - \Delta PS/2 – 2\text{-}1/2$ (inches)

whichever is greater, but not less than "a" required by Caltrans BDS.

ΔEQL = Longitudinal Displacement at ODE level.
EQU = Longitudinal Displacement at MDE level.
ΔT = total thermal displacement range.
ΔPS = total prestress shortening.
Round MR and "a" up to nearest 1/2 inch and 1/4 inch respectively.

3A9.7 Abutments, Piers, and Walls

At the ODE level, maintain an open gap longitudinally and provide full transverse load capacity. At the MDE level, consider the contribution of the approach slab in the longitudinal direction. The procedure is as follows:

For abutments, piers and walls, refer to Caltrans SDC.

3A9.8 Foundations

For both ODE and MDE, footings shall be provided with enough vertical carrying capacity within a 45° cone directly under the column, to carry the unfactored column dead load reaction.

3A9.9 Expansion Joint Hold-downs

For both ODE and MDE, hold down devices shall be provided at all supports and intermediate hinges where the vertical seismic forces (Load Case 1 and 2) oppose and exceed 50% of the dead-load reaction or Load Case 3 produces net uplift. The minimum seismic design force for the hold-down device shall be the greater of:

Load Case 1 & 2

10% of the dead load reaction or
1.20 times the net uplift force.

Load Case 3

Net uplift force

3A10.0 MINIMUM SEAT WIDTH

The seismic design displacements for determining seat width shall be the greater of either those obtained from analysis at the MDE level using spectra and effective column stiffness or as specified in Caltrans BDS implemented AASHTO LRFD Bridge Design Specifications, with California Amendments and AASHTO Guide Specifications for LRFD Seismic Bridge Design, latest editions.

3A11.0 RESTRAINERS

The detailed design of restrainers shall be based upon the philosophy and guidelines set forth in Caltrans BDS.

3A12.0 SEISMIC BASE ISOLATION

A base isolation system may be considered in the design of special bridges and aerial structures upon approval by Metro and should conform to the following subsections.

Design of all base isolated bridges and aerial structures shall conform to Caltrans BDS implemented AASHTO Guide Specifications for Seismic Isolation Design, latest edition with the following modifications.

3A12.1 Analysis Procedure

All base isolated bridges and aerial structures shall be ~~analyzed~~ **analysed** by the following two methods for both ODE and MDE and the most stringent case shall govern the design of the structural elements and isolation system.

Method 1: Response Spectrum Analysis

An equivalent linear response spectrum analysis shall be performed using the appropriate ground motion response spectra (horizontal and vertical) as defined in Chapter 2 of this criteria.

Method 2: Time-History Analysis

A non-linear time-history analysis of the combined structure and isolator system shall be performed. This method will incorporate the actual force deflection characteristics of the systems together with a minimum of three ground motion time histories that represent the seismicity of the site, and must be approved by Metro.

3A12.2 Isolation System

The isolation system shall be ~~analyzed~~ **analysed** using deformational characteristics. The isolation system shall be ~~analyzed~~ **analysed** with sufficient detail to:

- Account for the spatial distribution of isolator units.
- Calculate translation, in both horizontal directions, and torsion of the structure above the isolation interface, considering the most disadvantageous location of mass eccentricity,
- Assess overturning/uplift forces on individual isolator units, and
- Account for the effects of vertical load, bilateral load and/or the rate of loading if the force deflection properties of the isolation system are dependent on one or more of these attributes.

No tension is allowed in isolators.

3A12.3 Design Forces for Seismic Performance

The isolated structural above and below the isolated system shall be designed using all the provision for a non-isolated structure. The design and detailing of seismic isolation devices shall be designed in accordance with the provisions of Caltrans BDS and the AASHTO Guide Specifications for Seismic Isolation Design, whichever is more critical.

The seismic design force for columns and piers shall not be less than the forces resulting from a lateral force applied at the isolator location corresponding to the yield level of a softening system, or the static friction level of a sliding system, or the ultimate capacity of a sacrificial wind-restraint system.

3A12.4 Structure and Rail Interaction

Special analysis shall be performed to evaluate the interaction between the structural components and track work above it; special attention shall be given at the expansion joints and abutments. At ODE, no damage to the rails or no transverse residual gap between adjacent segments of rails is allowed. At MDE, the level and extent of the damage to the rails shall be defined.

3A13.0 Seismic Design for Ground and Embankment Stability

For seismic design for ground and embankment stability, apply the National Cooperative Highway Research Program Reports, Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments (TRB 2008, NCHRP 2008).

The seismic stability and potential permanent deformation of sloping ground or embankments supporting aerial guideway and bridges along proposed alignments shall be investigated. Investigations should included evaluation of the potential for ground liquefaction and related deformations. The evaluations and associated analyses should be displacement based leading to the determinations of potential lateral deformations of slopes or embankments and ground settlement. Total settlement and lateral ground deformations under ODE seismic events shall not be allowed to exceed 2 inches to allow for track re-leveling or re-alignment. Larger deformations may be allowed for MDE events on a case-by-case basis on approval by Metro.

The stability of slopes and embankments shall be evaluated using either (1) the seismic coefficient approach in a pseudo-static stability analysis or (2) the slope-displacement method as described in the NCHRP Project 12-70 Reports on the “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments,” (Transportation Research Board, 2008, NCHRP, 2008). If Method (1) quasi-static slope stability analyses lead to factors of safety less than 1.1, slope performance shall be evaluated using Method (2) where displacements are computed using Newmark time-history analyses. If computed displacements lead to unacceptable performance, appropriate mitigation measures hall be incorporated in the design.

If potentially liquefiable soils are identified along proposed alignments, liquefaction susceptibility shall be determined using the procedures documented in the AASHTO Bridge Design Specifications (AASHTO 2010, Article 10.5.4). The requirements for site investigation to assess liquefaction potential are described in Article 10.4 of the AASHTO Specifications. The liquefaction potential assessment should consider the impact of the following effects where liquefaction is judged to occur:

1. Loss of strength of liquefied layers (post liquefaction residual strength)
2. Flow failures, slope deformations
3. Post liquefaction ground settlement

The displacement performance of slopes and embankments underlain by liquefied soils may be evaluated in a similar manner to non-liquefiable cases, except residual strengths of liquefied soils are used in analyses (NCHRP, 2008, AASHTO, 2008). The post-liquefaction settlement of liquefied soil layers may be determined using procedures documented by Tokimatsu and Seed (1997)

For sites where liquefaction occurs around aerial structure or bridge foundations, structures should be ~~analyzed~~ **analysed** and designed for two configurations as documented in AASHTO, Article 10.5.4.2:

1. Non liquefied site soil configuration
2. Liquefied site soil configurations

For the latter case, residual strengths of liquefied soil layers are used for lateral and axial deep foundation response analyses. For those sites where liquefaction related permanent lateral ground displacements are determined to occur, the effects on pile performance should be evaluated. Downdrag forces on piles due to post liquefaction settlement should also be evaluated. If the above impact assessments yield unacceptable performance of the structures, appropriate mitigation measures shall be incorporated into the design.

REFERENCES

1. State of California Department of Transportation (Caltrans) Bridge Design Specification (BDS) referenced ”LRFD Bridge Design Specifications”, Fourth Edition, 2007, (Including 2008 and 2009 Interims), by the American Association of State Highway and Transportation Officials, (AASHTO).

2. State of California Department of Transportation (Caltrans), 2009, “Seismic Design Criteria, (SDC) Version 1.5.”

3. State of California Department of Transportation (Caltrans), August 6, 2009, Memorandum, “Implementation of Caltrans 2009 Seismic Design Procedure,” Kevin Thompson, James E. Davis, and Dolores Valls, Division of Engineering Services.

4. State of California Department of Transportation (Caltrans), August 12, 2009, Memorandum, “Quality Control/Quality Assurance for the 2009 Seismic Design Procedures,” Dolores Valls, Division of Engineering Services.

5. California Building Standards Commission, 2007, “California Building Code, California Code of Regulations, Title 24,” based on the 2006 International Building Code.

6. American Concrete Institute, 2008, “Building Code Requirements for Structural Concrete and Commentary”, (ACI 318-08, & ACI 318R-08.

7. American Institute of Steel Construction, 2005, “Code of Standard Practice for Steel Buildings and Bridges,” 13^{TH} Edition, (LRFD).

8. American Railway Engineering and Maintenance of Way Association (AREMA), 2008, “Manual for Railway Engineering.”

9. National Fire Protection Association, 2008, “Standard for Road Tunnels, Bridges, and Other Limited Access Highways (NFPA 502).”

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11. Applied Technology Council, 1996, “Improved Seismic Design Criteria for California Bridges: Resource Document,” Publication 32-1, Redwood City, California.

12. Seyed, M, April 1993, "Seismic Bridge Analysis Package", Caltrans Special Analysis Section.

13. Transportation Research Board, “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments,” NCHRP Report 611, 2008.

14. NCHRP, Project 12-70, “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments,” Draft Final Report Volume 2, Recommended Specifications, Commentaries and Example Problems, 2008.

15. Tokimatsu, K. and Seed, H.B., “Evaluation of Settlements in Sand due to Earthquake Loading,” Journal of Geotechnical Engineering, ASCE, Volume 113, No. 8, pp. 861-878, 1987.

CHAPTER 3

SUPPLEMENTARY SEISMIC DESIGN CRITERIA

Part B METRO SSDC FOR UNDERGROUND STRUCTURES

3B1.0 SCOPE

This Supplemental Seismic Design Criteria (SSDC) for Underground Structures replaces previous criteria (2003) used for Metro Gold Line Eastside Extension project. The criteria also replaces the "Supplemental Criteria for Seismic Design of Underground Structures" published in June 1984 by Metro Rail Transit Consultants.

The criteria and codes specified herein shall govern seismic design of Metro owned underground facilities including cut-and-cover subway structures, mined tunnels and stations, U-sections, shafts, earth-retaining structures, and other non structural and operationally critical components and facilities supported on or inside Metro underground structures.

These criteria address the general seismic design conditions that apply to the Metro Rail Project. Where there are cases of special designs encountered that are not specifically covered in these criteria, an appropriate technical source shall be determined and the appropriate procedure developed for the design.

3B2.0 DESIGN POLICY

The Metro Rail Project is a large-scale public project in an area highly susceptible to major earthquakes. Further, earthquake-initiated failures of selected structures and systems could lead to loss of life. For this reason Metro has developed special earthquake protection criteria for the project.

The guiding philosophy of earthquake design for the project is to provide a high level of assurance that the overall system will continue operating during and after an Operating Design Earthquake (ODE). Operating procedures assume safe shut down and inspection before returning to operation. Further, the system design will provide a high level of assurance that public safety will be maintained during and after a Maximum Design Earthquake (MDE). The definition of ODE and MDE levels is as follows:

The ODE is defined as the earthquake event which has a return period of approximately 150 years. Such an event can reasonably be expected to occur during the 100-year facility design life. The probability of exceedance of the ODE event is approximately fifty percent (50%) during the 100-year facility life.

The MDE is defined as the earthquake event which has a return period of approximately 2,500 years. Such an event has a small probability of exceedance during the 100-year facility life. The probability of exceedance of the MDE event is approximately four percent (4%) during the 100-year facility life.

3B3.0 PERFORMANCE OBJECTIVES

For the Operating Design Earthquake (ODE) which is likely to occur about once during the normal life expectancy, there shall be no interruption in rail service during or after the ODE. When subjected to the ODE, structures shall be designed to respond essentially in an elastic manner. There shall be no collapse, and no damage to primary structural elements. Only minimal damage to secondary structural elements is permitted, and such damage shall be minor and easily repairable. The structure shall remain fully operational immediately after the earthquake, allowing a few hours for inspection.

For the Maximum Design Earthquake (MDE) which has a low probability of being exceeded during the normal life expectancy, some interruption in rail service is permitted to allow for inspection and repairs following the MDE. When subjected to the MDE, it is acceptable that the structures behave in an inelastic manner. There shall be no collapse and no catastrophic inundation with danger to life, and any structural damage shall be controlled and limited to elements that are easily accessible and can be readily repaired. The structure should be designed with adequate strength and ductility to survive loads and deformations imposed on the structure during the MDE, thereby preventing structure collapse and maintaining life safety.

3B4.0 CODES, STANDARDS AND REFERENCES

The structural design for seismic loading shall meet applicable portions of the current editions of the codes, manuals or specifications identified in Section 5 - Structural/Geotechnical Criteria and those given below.

Unless otherwise noted herein, the relevant portions of the stated edition of the code or standard shall apply. If a new edition, interim specification or amendment is issued before the design is completed, the design shall conform to the new requirement to the extent practical, subject to Metro approval.

1. Technical Manual for Design and Construction of Road Tunnels – Civil Elements, March 2009 (FHWA-NHI-09-010), U.S. Department of Transportation, Federal Highway Administration, National Highway Institute.

2. Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankment, NCHRP Report 611, 2008 Transportation Research Board.

3. California Building Code, California Code of Regulations, Title 24, Part 2, California Building Standards Commission, based on the International Building Code. This code and its amendments is referred to herein as the Building Code.

4. Metro Rail Supplemental Seismic Design Criteria (Metro Rail SSDC), Chapter 2 – Seismic Design Ground Motion Criteria.

3B5.0 SEISMIC HAZARD AND DESIGN GROUND MOTION PARAMETERS

Seismic environment, seismic hazard analysis procedure and the design ground motion parameters for the Metro Rail Project are presented in Chapter 2 (Seismic Design Ground Motion Criteria) of the Metro Rail SSDC. For seismic design of underground structures, important seismic design ground motion parameters include (for both MDE and ODE) design earthquake magnitudes, design site-to-source distances, design response spectra, design ground motion time histories (spectrum-compatible), design ground motion peak values, design soil shear displacement (or shear strain) profiles, and design fault rupture displacements and other relevant parameters.

3B6.0 GENERAL DESIGN PROCEDURE

The general procedure for seismic design of underground structures is based primarily on the ground deformation approach specified herein. During earthquakes, underground structures move together with the surrounding geologic media. Therefore, the structures are designed to accommodate the deformations imposed by the ground taking into consideration the effects of soil-structure interaction.

Underground tunnel structures undergo three primary modes of deformation during seismic shaking: ovaling/racking, axial, and curvature deformations. The ovaling/racking deformation is caused primarily by seismic waves propagating perpendicular to the tunnel longitudinal axis. Vertically propagating shear waves are generally considered the most critical type of waves for this mode of deformation (Figure 3B-1). The axial and curvature deformations are induced by components of seismic waves that propagate along the longitudinal axis (Figure 3B-2).

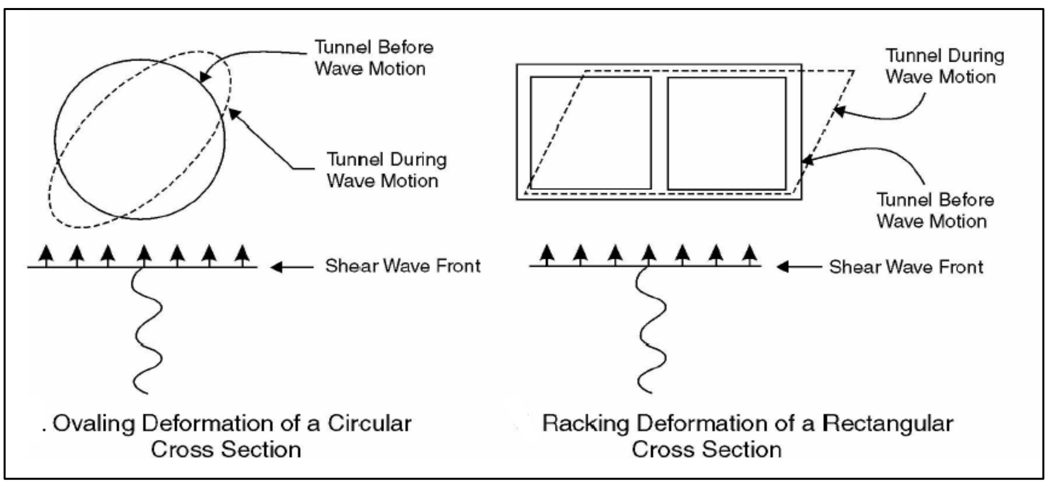


Figure 3B-1 Tunnel Transverse Ovaling and Racking Response to Vertically Propagating Shear Waves (Wang, 1993)

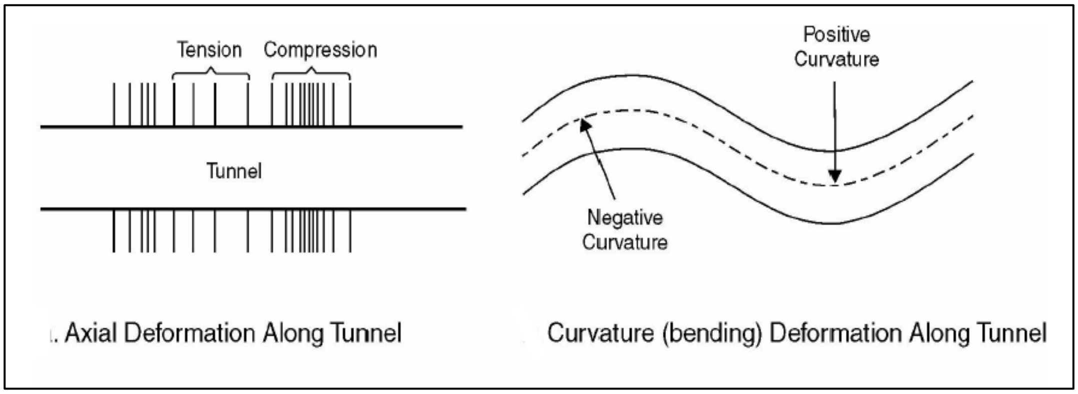


Figure 3B-2 Tunnel Longitudinal Axial and Curvature Response to Traveling Waves

3B7.0 BORED CIRCULAR TUNNELS

Bored circular tunnels include earth tunnel sections and rock tunnel sections, using either the precast concrete segmental lining or cast-in-place concrete lining. Design details for reinforced concrete tunnel lining shall be in accordance with the provisions of the California Building Code and Caltrans implemented AASHTO LRFD. Additional guidance in tunnel applications not generally covered in bridge and building codes is presented in FHWA-NHI-09-010 Report, "*Technical Manual for Design and Construction of Road Tunnels*", Chapter 10. The design shall also comply with the requirements specified in Metro Rail Design Criteria Section 5 – Structural/Geotechnical.

3B7.1 Seismic Demands due to Ovaling Deformations

There are two general approaches to determining the seismic deformation of circular tunnels.

The first approach is based on closed form solution that accounts for soil-structure interaction effect. The closed form solution is based on the following assumptions: (1) the tunnel is of completely circular shape (without decks or walls inside) with uniform lining section, (2) surrounding soil is uniform, and (3) there is no interaction effect from adjacent tunnels or other structures.

The second approach is a numeric**al** modeling approach that ~~relies on~~**uses** ~~mathematical~~ **numerical** models of the structures ~~(including adjacent structures if relevant)~~ to account for structural properties, varying soil stratigraphy and soil properties, **leading to more rigorous solutions for deformations**~~loadings and deformations more rigorously~~. ~~These structural models are generally run on computers with specialized software.~~ If the actual soil-structure systems encountered in the field are more complex than the assumed conditions described ~~above~~ for the closed form solution approach **(**which could lead to unreliable results**)**, then the use of the numerical modeling approach should be adopted.

3B7.1.1 Closed Form Solution

For the closed form solution the seismic ovaling loads for the lining of bored circular tunnels is defined in terms of change of tunnel diameter (ΔD_{EQ}) caused by the vertically propagating shear waves of the MDE and ODE ground motions. ΔD_{EQ} can be considered as seismic ovaling deformation demand for the lining. The procedure for determining ΔD_{EQ} is summarized as-follows:

1. Calculate the expected free field ground shear strains caused by the vertically propagating shear waves of the design earthquakes, for both MDE and ODE. The maximum free-field ground shear strains, $\gamma_{\max}$, shall be derived at the elevation of the tunnel section that is of interest. The determination of the maximum free-field ground shear strain, $\gamma_{\max}$, requires site-specific one-dimensional site response analyses using computer programs such as SHAKE 91 (Idriss and Sun, 1992). The acceleration time histories required for such analyses should be determined from spectral matching procedures (described in Chapter 2) where the "rock outcrop" spectra is defined by the MDE and ODE ground acceleration from the USGS 2009 PSHA Website (USGS 2009). In performing the site-specific site response analysis appropriate strain dependent shear modulus reduction curves and damping curves are to be assigned to site soil or rock strata at the site using accepted relationships such as those for cohesive soils (Vucetic Dobry, 1991) and sands (EPRI, 1993), and where the maximum shear modulus values are determined from measured in-situ shear wave velocities.

2. By ignoring the stiffness of the tunnel, which is applicable for tunnels in rock or very stiff/dense soils, the lining is assumed to conform to the distortion imposed on it by the surrounding ground with the presence of a cavity in the ground due to the tunnel excavation. The resulting diameter change of the tunnel is estimated as follows:

$$\Delta D_{EQ} = \pm 2\gamma_{max}(1-\nu_m)D$$

where:

ν_m = Poisson's ratio of the surrounding ground

D = diameter of the tunnel

3. If the tunnel is stiff relative to the surrounding soil, the effects of soil-structure interaction shall be taken into consideration. The stiffness of the tunnel relative to the surrounding ground is quantified by the flexibility ratio, F, and compressibility ratio, C, which are measures of the flexural stiffness (resistance to ovaling) and ring compression or extension stiffness, respectively, defined as follows:

$$F = \frac{E_m(1-\nu_c^2)R^3}{6E_cI_c(1+\nu_m)}$$

$$C = \frac{E_m(1-\nu_c^2)R}{E_ct(1+\nu_m)(1-2\nu_m)}$$

where:

E_m = strain compatible elastic modulus of the surrounding ground
E_c = elastic modulus of the concrete lining
R = nominal radius of the concrete lining
I_c = moment of inertia of the concrete lining (per unit width)
ν_c = Poisson's ratio of the concrete lining
ν_m = Poisson's ratio of the surrounding ground
t = thickness of the concrete lining

The strain compatible elastic modulus of the surrounding ground E_m shall be derived using the effective strain-compatible shear modulus G_m obtained from the results of the site-specific site response analysis.

The moment of inertia of the concrete lining I_c per unit width shall be determined based on the expected behavior of the selected lining under the combined seismic and static loads, accounting for cracking and joints between segments and between rings as appropriate. The cracked section of concrete shall be used for bending stress as appropriate.

4. Derive the tunnel diameter change, ΔD_{EQ}, accounting for the soil-structure interaction effects using the following equation:

$$\Delta D_{EQ} = \pm⅓ (K_1 F \gamma_{max} D)$$

where:

K_1 = seismic ovaling coefficient $= \dfrac{12(1-\nu_m)}{2F+5-6\nu_m}$

The seismic ovaling coefficient curves are presented in Figure 3B-3.

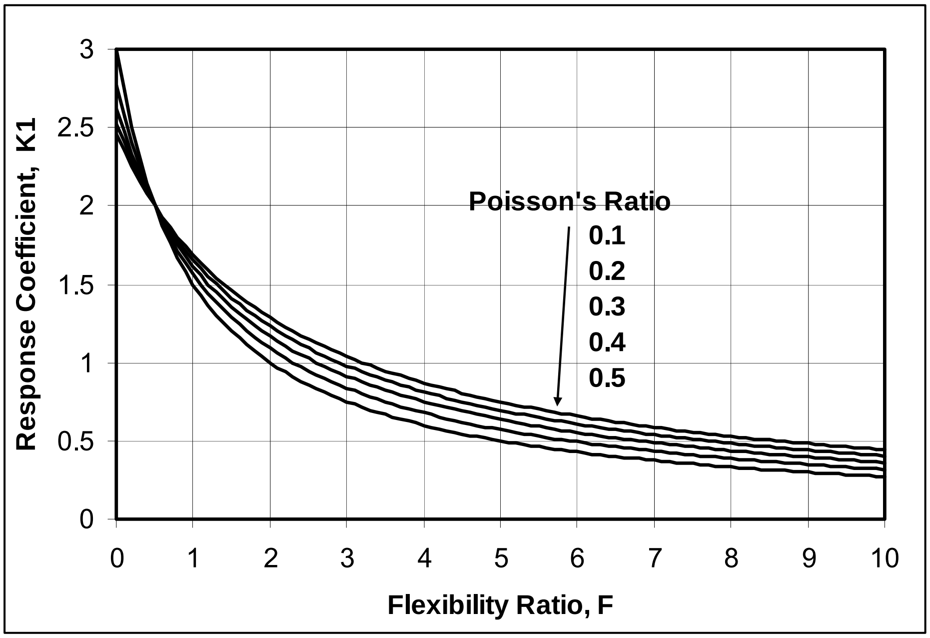


Figure 3B-3 Seismic Ovaling Coefficient, K_1

5. Derive the seismic loading effect, EQ, associated with the seismic ovaling deformation ΔD_{EQ} by using the loading combinations and load factors presented in Table 5-3 in Metro Rail Design Criteria, Section 5, Structural/Geotechnical:

If the tunnel lining is expected to behave in an essentially elastic manner and for lining that can be modeled with a uniform bending stiffness (E_cI_c), the internal seismic force EQ, expressed in terms of maximum thrust T_{max} per unit width and maximum bending moment M_{max} per unit width can be derived as follows:

$$T_{max} = K_2 \gamma_{max} \frac{E_m}{2(1+\nu_m)} R$$

$$M_{max} = \frac{1}{6} K_1 \gamma_{max} \frac{E_m}{1+\nu_m} R^2$$

where:

$$K_2 = 1 + \frac{F[(1-2\nu_m)-(1-2\nu_m)C] - 0.5C(1-2\nu_m)^2 + 2}{F[(3-2\nu_m)+(1-2\nu_m)C] + C(2.5-8\nu_m+6\nu_m^2) + 6 - 8\nu_m}$$

The resulting bending moment induced maximum fiber strain, ε_m, and the hoop force (i.e., thrust) induced strain, ε_T , in the lining can be derived as follows:

$$\varepsilon_T = K_2 \gamma_{max} \frac{E_m}{2(1+\nu_m)} \cdot \frac{R}{E_c t}$$

$$\varepsilon_m = \frac{1}{6} \cdot \frac{t}{2} K_1 \gamma_{max} \frac{E_m}{1+\nu_m} \cdot \frac{R^2}{E_c I_c}$$

The lining coefficient K_2, primarily used for the thrust response evaluation, is graphically presented in Figures B-4, B-5, and B-6 for Poisson's Ratio values of 0.2, 0.35 and 0.5, respectively.

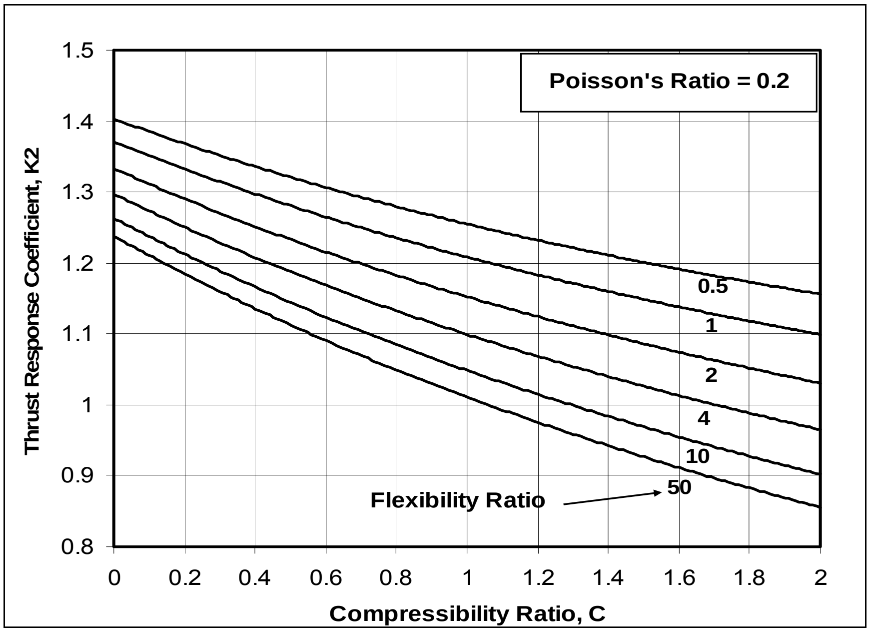


Figure 3B-4 Lining Response Coefficient, K_2, for Poisson's Ratio = 0.2 (Wang, 1993)

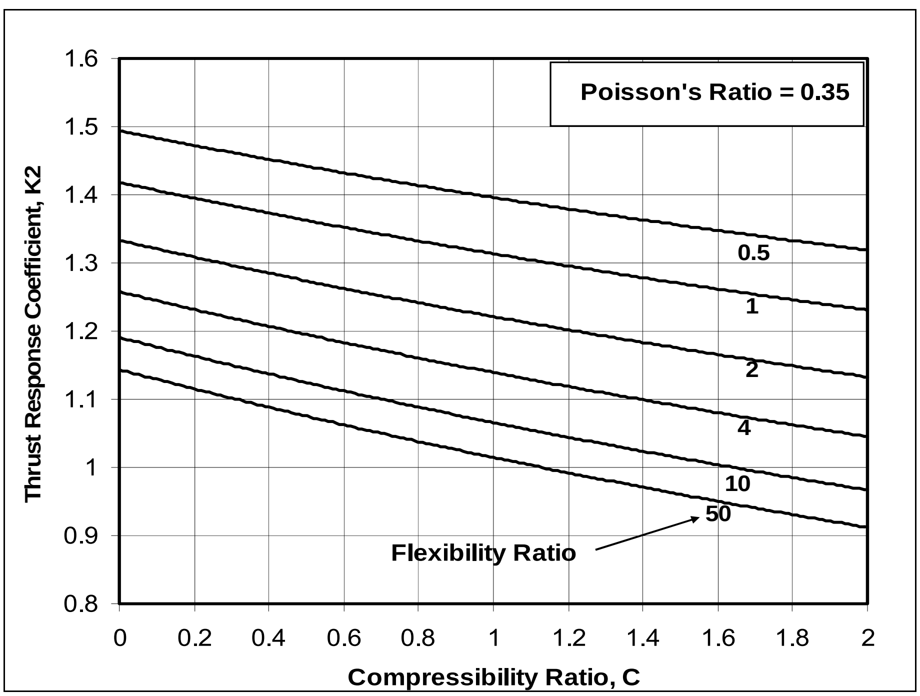


Figure 3B-5 Lining Response Coefficient, K_2, for Poisson's Ratio = 0.35 (Wang, 1993)

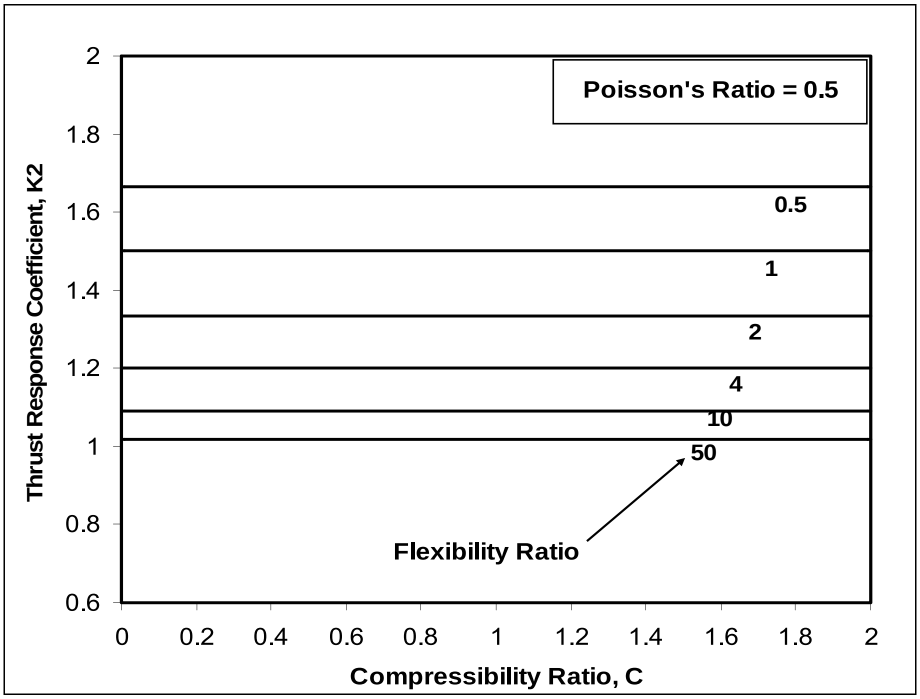


Figure 3B-6 Lining Response Coefficient, K_2, for Poisson's Ratio = 0.5 (Wang, 1993)

The equations for T_{max} and ε_T, are based on a no-slip condition at the soil/lining interface, where relative slip movement between the exterior side of the tunnel lining and the surrounding soil is assumed not to occur. This no-slip assumption produces more conservative results for evaluating T_{max} and ε_T. On the other hand, more critical results are obtained for M_{max} and ε_m, as expressed by the equations presented above, by assuming a full-slip condition at the soil/lining interface.

If inelastic displacement is anticipated to occur in the tunnel lining, such as under the MDE loading condition, the internal seismic force EQ must be carefully evaluated by considering the structural detailing of the tunnel lining (segments as well as segmental joints) and if necessary inelastic displacement-based structural analysis should be conducted to ensure the tunnel lining has adequate strength and ductility to accommodate the seismic ovaling deformation ΔD_{EQ}.

3B7.1.2 Numerical Modeling Approach – Ovaling Analysis

The actual soil-structure system encountered in the field for underground structures are often more complex than the ideal conditions described above and may require the use of numerical methods. This is particularly true in cases where a very important tunnel structure is located in a severe seismic environment.

For transverse ovaling analysis, a two-dimensional method of analysis is generally considered an adequate numerical modeling approach. The model needs to be developed with the capability of capturing soil-structure-interaction

(SSI) effects as well as appropriate depth-variable representations of the earth medium and the associated free-field motions (or ground deformations) obtained from two-dimensional site-response analyses of representative soil profiles.

In using numerical modeling methods to ~~analyze~~ **analyse a** bored tunnel cross-section subjected to ovaling deformation**,** the following considerations should be included:

1. As a minimum, ~~analyze~~ **analyse** the structure, surrounding ground, and seismic imposed deflections as a two dimensional soil-structure model ~~(see example of a continuum soil-structure model in Figure 3B-7)~~.

2. Include in the model, if relevant, the internal decks and walls to assess their effects on stress concentration and tunnel deformation ~~(Figure 3B-7)~~.

3. Model the effects of the liner joints, particularly where the joints are not properly restrained against opening and closing.

4. Accurately model the soil stratigraphy and soil properties and loads relative to the geotechnical profile and cross-section.

~~5. Apply the deformations due to the propagation of shear wave based on site-specific site response analyses for both the ODE and MDE. In general, the deformation analysis can be performed using pseudo-static or pseudo-dynamic analysis in which displacements or displacement time histories are statically applied to the soil-structure system. Dynamic time history analysis can also be used to further refine the analysis when necessary, particularly when some portion(s) of the tunnel structure can respond dynamically or under earthquake loading, i.e., in the case where the *inertial effect* of the tunnel structure is considered to be significant.~~

~~6.Evaluate the loads and deformation not only in the liner segments themselves but also at the joints.~~

The numerical methods used to evaluate the structural ovaling response of a tunnel structure to meet Metro performance requirements are documented below. The analysis method should evaluate the loads and deformations in liner joints and segments. Method 1 is the minimum design requirement for a numerical model approach.

1. Method 1: Pseudo-Dynamic Time-History Analysis

The pseudo-dynamic analysis consists of stepping the two-dimensional soil-structure system statically through displacement time-history simulations of free-field displacements obtained through a two-dimensional site-response analysis. The analysis entails two steps:

Step 1 – Determine time-histories of seismic displacements (horizontal and vertical components at node points) from a two-dimensional site-response analysis (using the QUAD4 finite element program, for example). This analysis approach is termed a "scattering" analysis where a cavity represents the tunnel (Figure 3B-7). The input acceleration time-histories

required for such analyses are described in Chapter 2 of the Supplemental Seismic Design Criteria.

Step 2 – Determine interface soil springs (normal and tangential components) to reflect the effects of soil-structure interaction (Figure 3B-7). Where appropriate tangential interface springs should allow for slippage, and normal interface springs for gapping.

Step 3 – Apply displacement demands (Step 1 – normal and tangential components) to the structure using a static stepping procedure, to determine the maximum structural response. To evaluate structural performance, seismic stresses should be superimposed on initial static stresses.

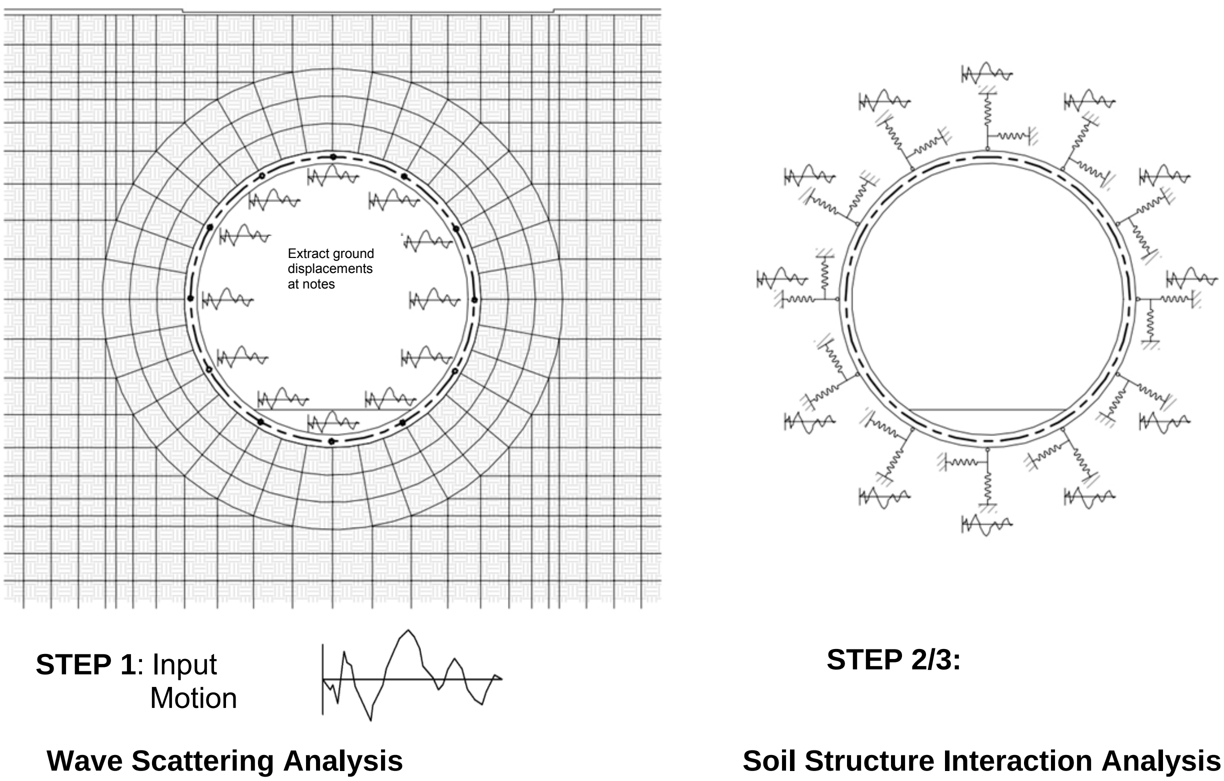


Figure 3B-7 Two-Dimensional Tunnel Analysis

2. **Method 2: Dynamic Time-History Analysis**

Where inertial effect from the tunnel structure are judged significant, the pseudo-dynamic static stepping procedure described above, should be replaced by an analysis procedure where the displacement time-histories are applied to the tunnel structure through the interface springs in a fully dynamic mode. Appropriate damping should be used. As is the case for Method 1, initial static loads must be considered.

3B7.1.3 Geological Variations

In choosing tunnel cross sections for analysis, particular attention should be placed on mixed face conditions and in changes in site conditions along the longitudinal axis of the tunnel. The interface soil springs shown in Figure 3B-7 should reflect changes in stiffness associated with geological variations.

3B7.2 Racking Response of Rectangular Tunnels

Shallow depth tunnels are often of rectangular shape and are often built using the cut and cover method. During earthquakes such box shaped tunnels will experience transverse racking deformations due to shear distortions of the ground, as shown in Figure 3B-1. Methods of analysis to determine seismic racking deformations are described in Section 3B8.0. Note that where compacted backfill soils are used for cut and cover structures, the effect of backfill cannot be accounted for using analytical closed-form solutions, and numerical analyses are required to determine racking displacement demands.

3B7.~~2~~3 Seismic Demands from Axial/Curvature Deformations

1. The evaluation procedures for the longitudinal response (due to axial/curvature deformations) of tunnel structures should be based on the procedures outlined in Section 13.5.2 of the Technical Manual for Design and Construction of Road Tunnels (FHWA-NHI-09-010 Report, 2009). The Free-Field Deformation procedure (in section 13.5.2.1 of the Road Tunnel Manual) may be used to determine the strains related to axial and longitudinal deformation of the tunnel under seismic ground motions **where uniform geological conditions exist and where the tunnel is sufficiently deep to avoid wave reflection from the ground surface**. ~~Supplement the analysis with Numerical Modeling Approaches similar to those in Section 13.5.2.3 of the Technical Manual where there are abrupt changes in structural stiffness or geological properties.~~

2. For the Free-Field Deformation analysis calculate the combined axial and bending strains from the P-Waves (pressure waves), S-Waves (shear waves), and R-Waves (Rayleigh waves) using the formulae given in Section 13.5.2.1 of the Technical Manual. The parameters associated with each class of wave are to be developed and provided by Project Geotechnical Engineers.

3. **Where uniform geological conditions do no exist,** ~~Use~~ **use** Numerical Modeling to investigate the effects of abrupt changes in structural stiffness or geological properties**, as described below**. Structural stiffness change locations

can include the tunnel breakouts at the portals; where egress and ventilation shafts may joint the tunnel; and other local hard spots. Geological changes requiring numerical modeling include area**s** of abrupt change in soil stiffness **or topographic conditions** along the alignment. ~~These include the interfaces between liquefiable and non-liquefiable soils and the interfaces between soil and rock.~~

3B7.3.1 Numerical Modeling Approach – Longitudinal Response

A numerical modeling approach for the longitudinal seismic response of a tunnel structure is desirable where tunnels run through highly variable subsurface conditions or have variable structural stiffness. Differential displacement demands during seismic events may generate significant axial, shear and bending stresses and stress concentrations within the structure, especially at interfaces along the tunnel. Three dimensional seismic response analyses to determine displacement demands and related interface spring supports are necessary to provide the means for structural analyses to beam-on-Winkler spring foundation theory. Three components of ground deformation should be determined for each support location, as shown in Figure 3B-8.

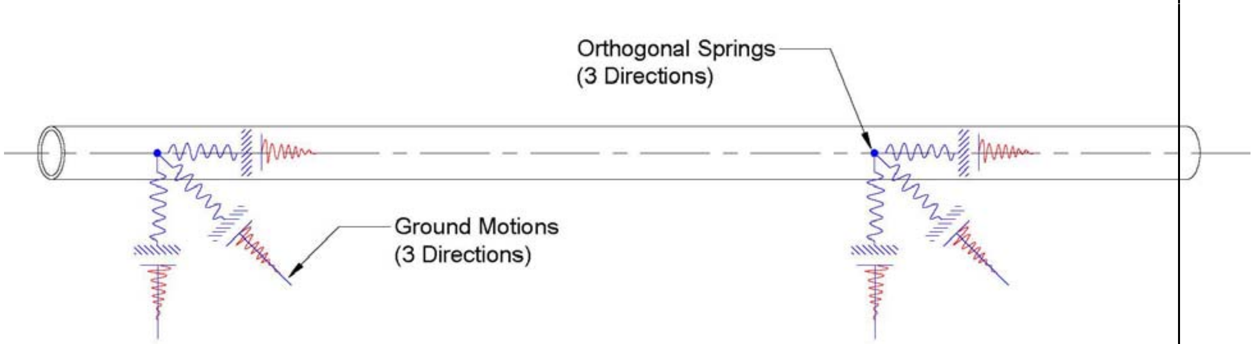


Figure 3B-8 Three Dimensional Axial/Curvature Analysis

The three dimensional site response analyses may be evaluated by uncoupling longitudinal and transverse seismic response. A two dimensional finite element or finite difference subsurface cross-section model through the longitudinal tunnel axis can be used to evaluate the seismic displacement time history response in the longitudinal and vertical directions. The effect of wave traveling/phase shift may also be included in analyses if determined to be significant. The transverse out of plane motions on the tunnel axis at selected cross sections may be determined using one dimensional site response programs such as SHAKE (Idriss and Sun, 1992) using identical subsurface properties as used for the two dimensional model. The properties of the interface springs should be consistent with soil modulus values used in seismic response analyses.

The evaluation of the longitudinal response of the tunnel structure may be determined by a three dimensional pseudo-dynamic time history analysis, where displacement time histories are applied in a statically stepping manner (analogous to the numerical ovaling analyses) to the structural model through the interface

springs. The resulting sectional forces and displacements in the structural tunnel element (as well as tunnel joints, if applicable) will reflect the axial/curvature deformation demands. Note that the articulation characteristics of the tunnel circumferential joints (between two adjacent tunnel rings) may play an important role in the longitudinal seismic response of a tunnel and hence should be considered in the structural model.

Where inertial loads on the tunnel structure are considered significant, the displacement time histories may be applied through the interface springs in a fully dynamic mode.

3B7~~.34~~Stability Checks

1. There are two levels of stability checks for the tunnel liner design based on the performance criteria for the ODE and MDE seismic ground motions. Under the ODE there will be no to minimal damage to the lining segments, joints, and water tightness. The tunnel will be able to be put back in service after a post earthquake inspection. Under the MDE the criteria are no collapse and being able to evacuate the tunnel safely immediately after the MDE. Inelastic deformations and damage are allowed but any structural damage shall be controlled and limited to elements that are easily accessible and can be readily repaired. No collapse mechanisms are allowed.

2. Combine the seismic demands from ~~the S-wave~~ ovaling, axial, and curvature deformations by **the** Square Root of the Sum of the Squares (SRSS) method **if closed form solutions are used**.

3. ~~Combine the s~~**S**eismic demands induced form the three modes of deformation during seismic shaking (i.e., ovaling, axial, and curvature deformations) ~~with the~~ **must include** static ~~demand for~~**loads on** the structure.

4. Check the section capacities relative to AASHTO LRFD as modified for tunnels in the Technical Manual for Design and Construction of Road Tunnels (Section 10.3.3).

5. Check the structure's stability relative to performance level and ductility with the following additional criteria. The concrete and steel strain limits apply to reinforced concrete lining. Where precast concrete lining are present, the concrete and steel strain limits apply to the body of the segments themselves. Separate criteria apply to the joints between the segments depending on the extent to which a ductile connection across the segments are made.

   a. For the ODE level ground motions, design the lining to respond essentially in an elastic manner with no ductility demand. Do not exceed a concrete compression strain of 0.001. Do not exceed a steel tensile strain of 0.002.

   b. For the MDE level design, inelastic deformations are allowed, but kept to the acceptable levels. Do not exceed a concrete compression strain of 0.002. Do not exceed a steel tensile strain of 0.006. For the MDE level design, the concrete strain may be allowed to exceed 0.002 but not to exceed 0.004 provided that the strain is predominantly in flexural mode.

c. For joints without being specifically designed as ductile connection, no uplift (zero tension) is allowed across the joint. Check joint shear capacity. Shear friction approach using the compressive load across the joint may be used to check the joints shear capacity. Check the joints compressive and bearing capacity relative to unreinforced concrete, unless specifically designed bearing plates and confinement is provided.

d. If segment joints are specifically designed as ductile connection, the concrete and steel strain limits given above may apply to the joints. Design the connection so the steel crossing the joint develops 1.25 times the yield strength of the steel without brittle failure of the concrete at the anchorages.

3B7.45Interfaces

Interfaces between the bore tunnel and the more massive structures shall be designed as flexible/expansion joints to accommodate the differential movements **or as continuous structures to withstand applicable seismic forces**. The design differential movements~~ shall be determined by the Designer in consultation with the Project Geotechnical Engineers~~ **for flexible/expansion joints on the seismic force/moment for continuous interfaces shall be determined from the seismic soil-structure interaction analyses described above**.

~~3B7.5 Geological Variations~~

~~Abrupt changes in stiffness of geologic formations shall be accommodated by designing the structures in these formations for the static and seismic loads and deformations resulting from such variations. The design parameters for these conditions will be established on a case-by-case basis by the Project Geotechnical Engineer.~~

~~The effects of abrupt changes in stiffness of geological formations are important when tunnels are in mixed face conditions, passing longitudinally from a rock formation into a soil formation or from a very stiff formation into a very soft formation. The focus in this case is on the longitudinal response of the tunnel to the differential free-field deformations between the soil (a soft formation) and the rock (a stiff formation). The most critical mode of the differential free-field deformations (along the longitudinal axis of the tunnel) is the lateral differential deformations caused by the vertically propagating shear waves (i.e., spatially varying ground motion effects due to different site conditions).~~

~~The general procedure used for evaluating the effects of differential lateral free-field deformations on the longitudinal tunnel response in mixed face conditions due to the vertically propagating shear waves is summarized as follows:~~

~~1. Establish the free-field lateral soil and rock deformations along the tunnel alignment in the mixed face area. The free-field deformation profile along the~~

~~tunnel alignment can be developed by performing multiple site-specific one-dimensional site response analyses at various locations along the tunnel alignment to account for the spatially varying ground motion effects. The site-specific analyses can be performed using site response analysis programs such as SHAKE 91 (Idriss and Sun, 1992). Refer to Section B7.1.1 for more discussions on site-specific site response analyses.~~

~~2. Derive the non-linear lateral soil or rock springs along the longitudinal alignment of the tunnel structure to represent the varying ground stiffness to be used in the mixed face area in the subsequent soil-structure interaction analysis.~~

~~3. Develop a structural model based on the properties and geometry of the tunnel structure. The articulation characteristics of tunnel circumferential joints (between two adjacent tunnel rings) may play an important role in the longitudinal seismic response of a tunnel and hence should be considered in the structural model if applicable.~~

~~4. The differential lateral free-field deformation distribution along the length of the tunnel in the mixed face area (derived from Step 1 above) is then applied to the tunnel structure model (from Step 3) through the use of equivalent soil or rock springs (from Step 2) to account for the ground-structure interaction effect.~~

~~5. The seismic demands in terms of deformations and internal forces computed from the analysis (Step 4) shall then be checked against the capacity of the tunnel structure with particular focus on the details at the circumferential joints to accommodate the required deformation and force demands.~~

3B8.0 REINFORCED CONCRETE BOX AND STATION STRUCTURES

Reinforced concrete box structures include box (rectangular) cut-and-cover structures including passenger stations, and mined station sections that behave in similar manner as a rectangular structure during earthquake shaking. Design details for reinforced concrete box structures shall be in accordance with the provisions of the California Building Code and Caltrans implemented AASHTO LRFD. The design shall also comply with the requirements specified in Metro Rail Design Criteria Section 5 – Structural/Geotechnical.

For ODE and MDE design of reinforced concrete underground box structures use the Caltrans Seismic Design Criteria, and ACI 318 latest edition, with Metro specified rail transit loading. These are referred to throughout these criteria as “Caltrans SDC” and “ACI”, and are to be used in conjunction with the Extreme Event I (MDE) and IA (ODE) Load Combinations per Metro Rail Design Criteria Section 5.4.7.

Commentary: Note that load factors in the Criteria for Strength load combinations are based on AASHTO and therefore member capacities which are compared with those demands should be evaluated using AASHTO methods for consistency. Load factors in the Criteria for Extreme events (ODE/MDE) are 1.0 which indicates a limit state evaluation such as per Caltrans SDC is to be performed and consequently the calculated demands are independent of the capacity methodology used. The designer *should use a capacity methodology appropriate to the expected material behavior at the given demands.*

Seismic design of the transverse cross section of a structure shall consider two loading components:

1. The racking deformations due to the vertically propagating shear waves, which are similar to the ovaling deformations of a circular tunnel lining (see Figure 3B-1 in Section **3**B6.0).

2. Inertia forces due to vertical seismic motions.

3B8.1 Seismic Demands due to Racking Deformations

Two general approached can be used to determine the seismic racking deformation of rectangular box structures.

The first approach is based on semi-closed form solution that has been calibrated with a series of numerical analyses for a number of soil-structure configurations. The semi-closed form solution is based on the following assumptions: (1) the **structure** ~~tunnel~~ is of rectangular shape, **and** (2) **the** surrounding soil is reasonably uniform, and (3) there is no interaction effect from adjacent tunnels or other structures. **If the actual soil-structure interaction systems encountered in the field are more complex than the assumed conditions (which could lead to unreliable results), then the use of a numerical modeling approach should be adopted.**

The second approach is a numeric**al** modeling approach that ~~relies on~~**uses non-linear** ~~mathematical~~ **numerical** models of the ~~structures~~**structural**~~,~~ ~~(including adjacent structures if relevant)~~ to account for structural properties, varying soil stratigraphy and soil properties, **leading to more rigorous solutions for deformations**~~loadings and deformations more rigorously~~. ~~These structural models are generally run on computers with specialized software. If the actual soil-structure systems encountered in the field are more complex than the assumed conditions described above for the semi-closed form solution approach, leading to unreliable results, then the use of a numerical modeling approach should be adopted.~~

3B8.1.1 Semi-Closed Form Solution

The seismic racking loads for the lining of rectangular box structures are defined in terms of the **lateral** ~~sideway~~ racking displacements caused by the vertically propagating shear waves of the MDE and ODE ground motions. The differential **lateral** ~~sideway~~ racking displacement between the top and bottom elevations of a box structure is graphically shown as Δ_s in Figure 3B-~~8~~**9**. The internal forces and ductility demands due to the seismic racking deformation, Δ_s, can be derived by imposing the differential **lateral** deformation on the structure in an elastic or inelastic frame analysis. The procedure for determining Δ_s, for both MDE and ODE level design and with the consideration of soil structure interaction effects, is as follows:

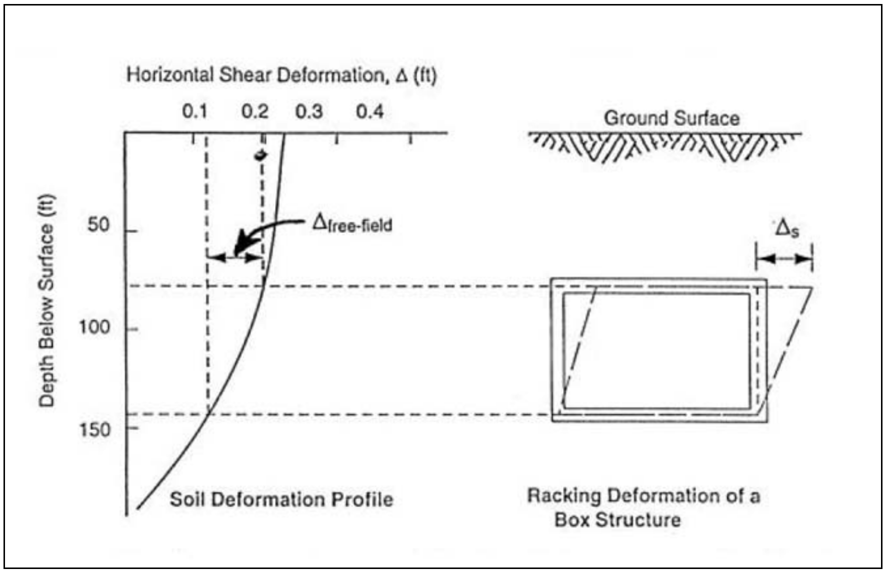


Figure 3B-~~8~~ 9 Free-field Soil Shear Displacement Profile and Racking Deformation of a Box Structure (Wang, 1993)

1. Calculate the **maximum** expected free field ground shear displacement profile caused by the vertically propagating shear waves of the design earthquakes, for both MDE and ODE (see Figure 3B-~~8~~9). The development of the free-field ground shear displacement profile requires site-specific one-dimensional site response analyses using computer programs such as SHAKE 91 (Idriss and Sun, 1992). Refer to Section B7.1.1 for more discussions on site-specific site response analyses.

2. Determine $\Delta_{free\text{-}field}$, the differential free-field shear displacements corresponding to the top and the bottom elevations of the box structure (see Figure 3B-~~8~~9).

3. Determine the racking stiffness, K_s, of the box structure by performing a structural frame analysis. The racking stiffness can be computed using the displacement of the roof subjected to a unit lateral force applied at the roof level, while the base of the structure is restrained against translation, but with the joints free to rotate. The ratio of the applied force to the resulting lateral displacement yields the racking stiffness K_s. In performing the structural frame analysis, the moment of inertia of the structural element I_c (for walls, floors, roof and invert slabs) per unit width shall be determined based on the expected behavior of each element under the combined seismic and static loads, accounting for cracking. The effects of potential development of hinges shall also be considered in the frame analysis.

4. Determine the flexibility ratio, F_r ,of the proposed design of the structure using the following equation:

$F_r = (G_m / K_s) \cdot (w/h)$

where:

K_s = racking stiffness of the box structure

w = width of the box structure

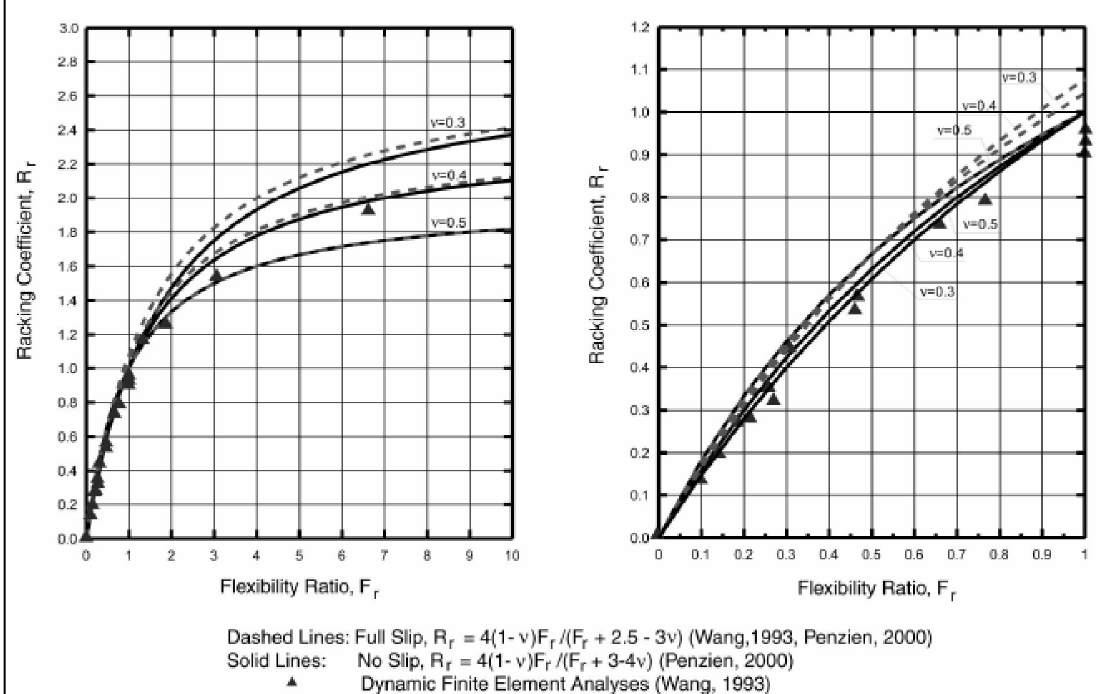
h = height of the box structure

G_m = average strain compatible shear modulus of the soil/rock layer between the top and bottom elevation of the structure. The average strain compatible shear modulus shall be derived based on the results of site-specific site response analyses

5. Based on the flexibility ratio obtained above, determine the racking ratio, R_r, for the proposed structure using Figure 3B-~~9~~10, or

$$R_r = \frac{4(1-\nu_m)F_r}{3-4\nu_m+F_r}$$ for no-slip interface condition (between soil and structure)

$$R_r = \frac{4(1-\nu_m)F_r}{2.5-3\nu_m+F_r}$$ for full-slip interface condition (between soil and structure)



a. Flexibility Ratios Between 0.0 and 10 b. Flexibility Rations Between 0.0 and 1.0

Figure 3B-~~9~~ 10 Racking Ratio Coefficient R_r for Rectangular Box Structures (MCEER-06-SP11, Modified from Wang, 1993, and Penzien, 2000)

6. Determine the racking deformation of the rectangular box structure, Δ_s, using the following relationship:

$$\Delta_s = R_r \cdot \Delta_{free-field}$$

7. The seismic demand (due to racking deformation) in terms of internal forces as well as material strains are calculated by imposing Δ_s upon the structure in a frame analysis (elastic or inelastic) as depicted in Figure 3B-~~10~~**11**.

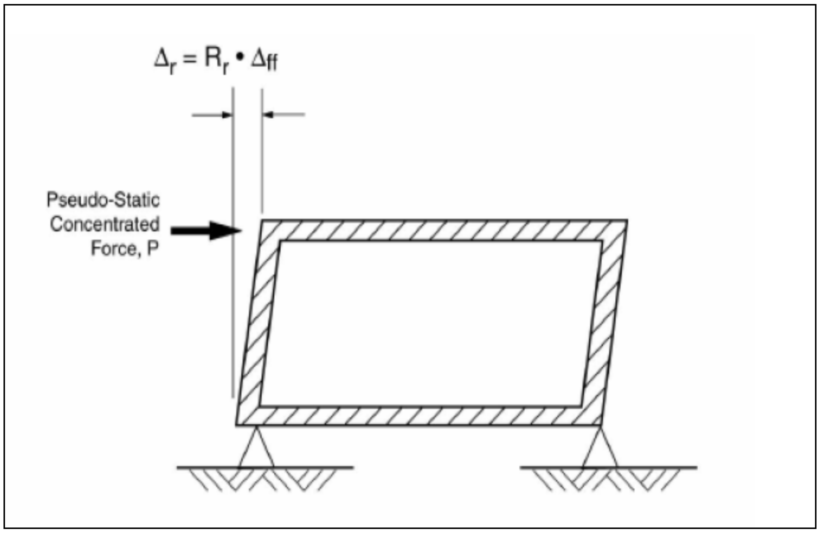


Figure 3B-~~10~~11 Simplified Racking Frame Analysis of a Rectangular Box Structure (MCEER-06-SP11, Modified from Wang, 1993)

3B8.1.2 Numerical Modeling Approach

In using numerical modeling methods to ~~analyze~~ **analyse** rectangular box **or station** structures subjected to racking deformation the following considerations should be included:

1. As a minimum, ~~analyze~~ **analyse** the structure, surrounding ground, and seismic imposed deflections as a two dimensional soil-structure model.

2. Include in the model, if relevant, the internal floors/decks and walls to assess their effects on stress concentration and ~~tunnel~~**structural** deformations.

3. Use appropriate assumption**s** in modeling the connections between the walls and the roof or invert slabs.

4. Accurately model the soil stratigraphy and soil properties and loads relative to the geotechnical profile and cross-section.

5. Apply the deformations due to the propagation of shear waves based on site-specific site response analyses for both the ODE and MDE **using an appropriate modeling approach as described below**. ~~In general, the deformation analysis can be performed using pseudo-static or pseudo-dynamic analysis in which displacements or displacement time histories are statically applied to the soil-structure system. Dynamic time history analysis can also be used to further refine the analysis when necessary, particularly when some portion(s) of the tunnel structure can respond dynamically or under earthquake loading, i.e., in the case where the *inertial effect* of the tunnel structure is considered to be significant. Figure 3B-11 is an example illustrating the two dimensional dynamic time history model for a cut-and cover tunnel structure.~~

3B8.1.3 Numerical Deformation Analysis Methods

The method used to evaluate structural response to the seismic displacement demands, depends on the extent to which an accurate assessment of the level of damage to the structure is required for MDE events (Section 3B3.0), in order to meet the Metro performance requirements. Potential analysis methods are documented below. Method 1 is a minimum design requirement for rectangular tunnels and stations. Method 3 is a minimum design requirement for non-rectangular stations.

1. Simplified pseudo-static method.
- **Step 1: Obtain maximum horizontal displacement demand profile over the structure height by evaluation of displacement time history "snapshots" determined from one-dimensional site response analyses. (See Section 2.3.3). For non-uniform site soil conditions or where interfaces between alluvium and bed rock occur over the structure height, the maximum displacement profile will not necessarily be triangular as shown in Figure 3B-8, particularly for non-uniform stratigraphy.**
- **Step 2: Apply horizontal maximum displacement demands to the structure through equivalent linear soil springs at structural node points (push over analysis) to determine structural response under seismic loading. Soil spring values should be chosen to reflect the effects of soil-structure interaction. Note: To evaluate structural performance, seismic stresses should be superimposed on initial static stresses.**

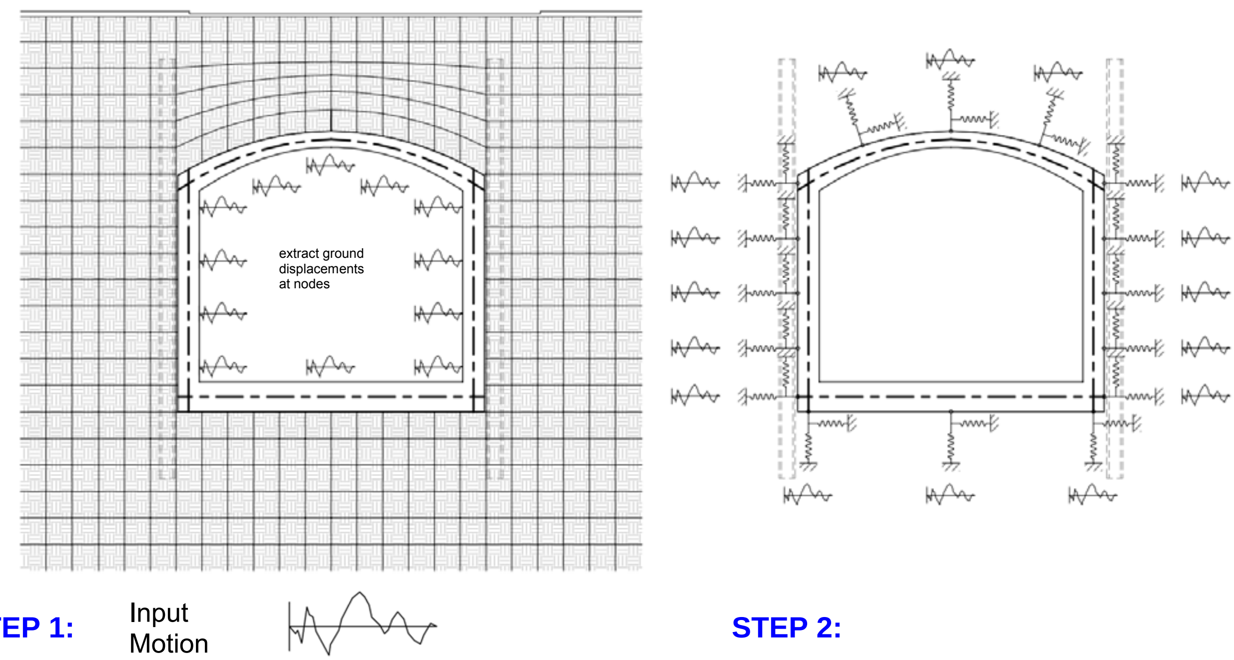
2. Simplified dynamic method.
- **Where inertial effects from the structure are judged significant, a dynamic time history analysis of the structural response should be undertaken using the horizontal displacement time histories determined above, input through the interface soil springs.**

3. Two dimensional dynamic method.
- **Step 1: Determine time histories of seismic displacements (normal and tangential components at node points) around the structure from two dimensional site response analysis (using the QUAD4 finite element program for example). This analysis approach is termed a "scattering" analysis, where a cavity represents the structure (Fig 3B-12). Input**

earthquake ground motions for response analyses are those used for one-dimensional analyses.

- Step 2: Apply displacement demand time histories to structure (normal and tangential) through soil interface springs to determine structural response under seismic loading, including inertial loading from the structure. Tangential interface springs should include a slip element to reflect a frictional component of interface behavior. Seismic stresses should be superimposed on initial static stresses to evaluate structural performance.

4. Fully coupled two dimensional dynamic finite element or finite difference method.
   - Using 2D computer programs such as FLAC or ADINA for example, determine the fully coupled dynamic response of the soil and structure, using nonlinear models for both soil and the structure as illustrated in Figure 3B-12. Appropriate bonding conditions should be included in the analyses



STEP 1: **STEP 2:**

Wave Scattering Analysis **Soil Structure Interaction Analysis**

Figure 3B-12a Two Dimensional Analysis Methods

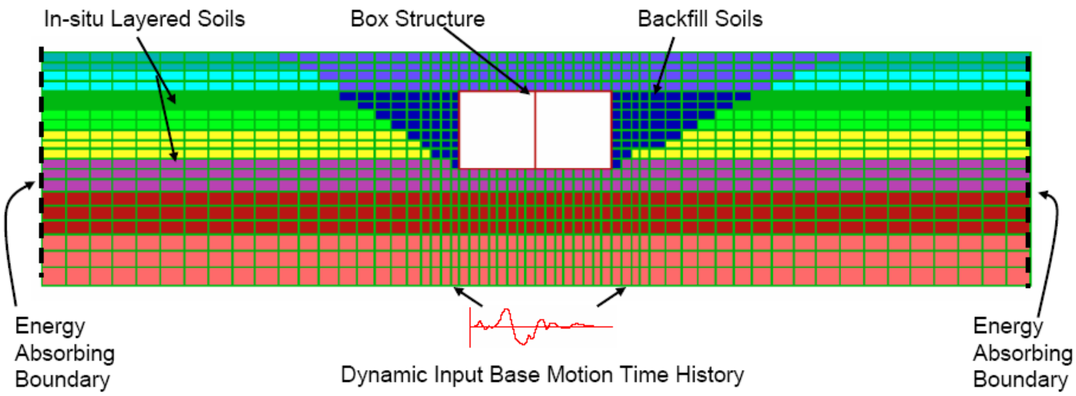


Figure 3B-~~11b~~ 12b ~~Sample~~ Finite Element Dynamic Time History Analysis Model

3B8.2 Seismic Demands due to Vertical Ground Motions

The effect of vertical seismic motions shall be considered for rectangular box structures. For structures constructed using cut-and cover methods the effect can be accounted for by applying a vertical pseudo-static loading, equivalent to the product of the vertical seismic coefficient and the combined dead and design overburden loads used in static design. For structures constructed using mining technique, the vertical pseudo-static loading can be estimated to be the product of the vertical seismic coefficient and the combined dead load and the weight of the loosened zone above roof, which shall be determined by Project Geotechnical Engineers. The vertical seismic coefficient can be reasonably assumed to be two-thirds of the design peak horizontal acceleration divided by the gravity. This vertical pseudo-static loading shall be applied by considering both up and down direction of motions, whichever results in a more critical load case shall govern.

Seismic demands due to racking deformations and vertical seismic motions are then combined by Square Root of the Sum of the Squares (SRSS) method.

3B8.3 Stability Check

1. Check the structure's stability based on the performance criteria for the ODE and MDE seismic ground motions.

2. Evaluate the possible mechanisms for MDE conditions (see Figure 3B-~~12~~**13**). Conditions with only two hinges in any one member, such as illustrated in Figure 3B-~~12a~~**13a**, are acceptable because a failure mechanism has not formed. Conditions with four hinges, such as illustrated in Figure 3B-~~12b~~**13b**, are also considered acceptable provided that the ground surrounding the structures is stable (i.e., no liquefaction or slope instability issues) because collapse is prevented by the surrounding materials. However, formation of any of mechanisms such as 1, 2, 3, 4, or 5 in Figure 3B-~~12c~~ **13** would lead to stability problems and these mechanism are, therefore, not acceptable.

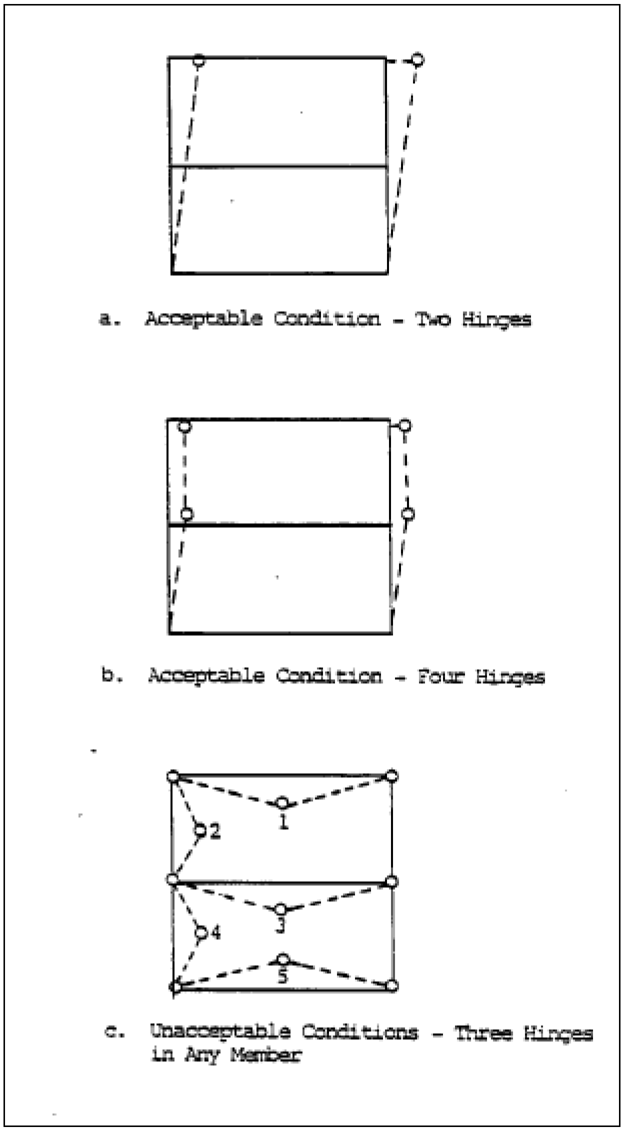


Figure 3B-~~12~~13 Structure Mechanisms under MDE

3B8.4 ODE Design Approach

Member capacities shall be evaluated using LRFD resistance factors (Φ), and required demands as determined from elastic analyses shall remain at or below the design capacity of the section (Ru ≤ ΦRn, which is a demand to capacity ratio D/C ≤ 1.0).

Nonlinear methods shall be used to study soil-structure interaction and P-Delta effects. Provide special consideration to areas of discontinuity, sudden changes in loading conditions, etc.

Box structural elements shall be designed to fail in a predominantly flexural mode by requiring the design shear capacity at each end to exceed the greater of the factored shear demand determined by analysis or the force required to develop the overstrength moment of the weaker member framed into a joint. Non-ductile failure modes shall be avoided.

While limit state analyses and special detailing are not required for this level of demand, members shall still be proportioned such that the box walls will reach a limit state prior to the box roof slab, box invert slab, and box joints. Reasonable efforts shall be made to proportion members such that global failure modes are ductile.

Commentary: For ODE level ground motions, this approach may be reasonably expected to allow the box structure primary members to perform essentially in an elastic manner with no ductility demand. Elastic structural analysis models are generally adequate for evaluating Gravity and ODE demands on the box structure. Note that compression forces may not be considered in calculating the shear capacity of the roof slab and invert slab per Criteria Section 5.4.7.C. The intent of the Criteria is for the designer to use engineering judgment to consider whether the use of shear or flexural capacity increases with compression loads which may otherwise be allowed by the reference codes is conservative for the loading conditions and elements being evaluated as the codes were not expressly written for below grade concrete structures.

3B8.5 MDE Design Approach

When MDE demands are relatively low, the analysis and design may follow the approach given in section 3B8.4 for ODE with member flexural and axial capacities evaluated using the nominal capacity Mn (Φ = 1.0 for bending and axial) and member shear capacities conservatively based on the design capacity ΦVn (Φ ≤ 0.9 for shear).

For the MDE level design, inelastic deformations are allowed, but kept to acceptable levels. When MDE analysis indicates relatively high demands such that inelastic behavior can be expected or where elastic design is not economical the following sections shall apply.

3B8.5.1 Analysis

Elastic structural analysis models shall be considered adequate for evaluating MDE cases which generate member flexural demands less than the nominal moment capacity based on expected material properties (Mne defined per Caltrans SDC), which may be estimated as 1.1·Mn when subjected to axial compression loads less than 0.1·Ag·f'c. For members subject to tension or high axial compression, this value shall be determined from a moment-curvature analysis of the section. Significant inelastic behavior is anticipated beyond Mne, therefore ductility and moment redistribution must be evaluated using inelastic analysis models which account for material nonlinearity.

Inelastic modeling shall include the effects of inelastic/plastic hinge zones, with properties based on expected material properties and strains outlined in Caltrans SDC for Pier Walls loaded in their weak direction. Instead of the bilinear elastic/perfectly-plastic hinge given by Caltrans SDC, the hinge model shall include the effect of post yield stiffness prior to reaching a perfectly-plastic region.

Commentary: It is important to note that the basic Caltrans SDC procedure is based on large levels of ductility demand (drift) while the typical below grade box structure may be expected to have a performance goal with relatively low levels of ductility demand. The full plastic moment capacity of a section as calculated using the Caltrans SDC may not develop until well beyond the strain limits specified. The inclusion of a post-yield relationship in the hinge definition is therefore necessary to allow the model to capture material stress/strain distributions at demands above the expected yield point which occur before reaching the Caltrans SDC plastic moment.

For non-linear inelastic time history analyses where the application of vertical ground motion seismic demands using SRSS per Section 3B8.2 is impractical, the effects of vertical and horizontal seismic ground motion may be applied using the alternative combination EQ = 100% EQhoriz +/- 35% EQvert.

3B8.5.2 Capacity Evaluation

Members designed to perform beyond a yield limit state shall be evaluated using a moment-curvature analysis program in a method consistent with Caltrans SDC to determine the approximate strain distribution in the concrete and reinforcement components as well as the member ductility for inelastic demand. The design shall consider the effects of both Lp per Caltrans SDC Section 7.6.2(a) as an upper bound and the lesser of Lp,min or h/2 as a lower bound unless justification of a larger value can be made.

Commentary: The idealized analytical plastic hinge length Lp per the Caltrans SDC procedure is typically longer than h/2, and is always longer than Lp,min. At low levels of inelastic demand the length of the yielding region is expected to be shorter than Lp, and its use would result in the calculation of unconservative total curvature and strain demands (due to their inverse relationship). There is limited research on hinge lengths for low levels of deformation, the h/2 lower bound is per FEMA 356 and is assumed to be conservative. It is not the intent of this section to reduce the dimensions of the plastic hinge zone for reinforcement detailing purposes.

3B8.5.3 Design

The Caltrans SDC and ACI seismic design basis is such that certain portions of members will be subjected to significant inelastic deformations, with flexural demands at those regions based on the plastic moment (Mp). Adjacent members which are to remain essentially elastic are defined as “capacity protected” and shall be designed to resist an overstrength demand moment. Where it may be shown that MDE demands are less than Mp, capacity protected components may be designed for an overstrength demand of 1.2 times the maximum demand obtained by inelastic analysis (1.2Mu) and not less than the nominal moment capacity based on expected material properties (Mne).

Commentary: The definition of Mp is different between ACI and Caltrans SDC, and may be determined per either method at the designer’s discretion. The design and detailing requirements are expected to be applied consistent with the selected method throughout, do not mix and match the two procedures. Note that Caltrans SDC is based on a premise that inelastic behavior occurs only in the vertical column/wall elements (capacity protected superstructure and foundation), while ACI is based on inelastic behavior only in beam elements (strong column/weak beam), the requirements and terminology should be translated to the appropriate box configuration and elements per the performance goals in this Criteria.

Where inelastic behavior is expected to occur in a member adjacent to a slab-wall joint at MDE, the joint shall be designed as a capacity protected component. The design shall consider the forces imposed on a corner joint based on the overstrength moment of the weaker member framed into it. This will force inelastic behavior out of the joint and into the adjacent member where damage may be more readily observed and repaired.

3B8.5.4 Performance Criteria

Commentary: Section 3B3.0 describes several performance objectives for MDE, the following criteria are provided as one approach which may reasonably be expected to achieve these performance goals. Alternative criteria may be submitted with appropriate justification for Metro approval.

Inelastic behavior shall be designed to occur in locations which are readily observable and accessible for repair. Cracking of box joints and concrete spalling at the exterior of the box may not be observable or readily repairable and should therefore be avoided. Damage to the box roof slab may cause undue concern of collapse and should also be avoided.

The box roof slab, box invert slab, and box joints shall be considered capacity protected to perform as essentially elastic with MDE demands. Box walls, interior floor slabs and columns shall be designed and detailed for ductile behavior to accommodate inelastic hinging, with a minimum local displacement ductility capacity of $\mu C \geq 4$ as calculated per Caltrans SDC. MDE global displacement ductility demand μD on the inelastic cross section should be less than 4.

Commentary: The global displacement ductility demand is calculated per Caltrans SDC by dividing the ultimate displacement by the initial displacement at which the first hinge *forms. Note that high gravity induced flexural demands at box joints may cause inelastic behavior during seismic racking to form quickly under low displacement. This will result in higher (more conservative) ductility demand values than for a similar above-ground structure designed per Caltrans SDC, therefore the given performance ductility demand is to be used as a guideline only. Ductility demands exceeding this value may indicate yielding behavior occurs at low levels of seismic demand and implies greater risk of damage.*

Members subject to MDE demands which exceed their nominal flexural capacity as calculated using expected material properties (Mne), shall have plastic rotation and axial demands determined by inelastic nonlinear analysis. Material strains shall then be evaluated at the plastic rotation and axial demands by using a nonlinear moment-curvature fiber section analysis.

The following strain limits are provided for control of damage where inelastic behavior is allowed:

Continuous elements with seismic cross-tie confinement:

Maximum steel reinforcement strain for reparability:	0.02
Maximum concrete strain at extreme fiber for reparability:	0.0033

Elements adjacent to discontinuities with seismic hoop confinement:

Maximum steel reinforcement strain for reparability:	0.025
Maximum concrete strain at extreme fiber for reparability:	0.004

Commentary: The reinforcement strain limit is intended to allow minimal to moderate amount of inelastic deformation of the steel reinforcement while avoiding bar buckling and fatigue failure. The concrete strain limit for continuous elements is intended to provide a reasonable control against extensive spalling of the cover and is based on two-thirds the ultimate unconfined concrete strain per Caltrans SDC. Note that concrete strain does not need to be checked for the confined portion since the strain limit at the extreme fiber (cover) will control. It is recognized that elements adjacent to openings in the box will be subject to higher demands with a corresponding increased risk of damage, however it should be confined to localized regions.

3B8.6 Detailing

Detailing of the box walls, floor and roof for inelastic behavior at MDE (D/C ratio exceeds 1.0) shall be per ACI 318 Chapter 21 and as modified by this Criteria. A minimum of two layers of reinforcement shall be used. Sufficient cross-ties shall be provided to prevent longitudinal bar buckling and comply with the confinement requirements in plastic hinge zones and joints. Cross ties in plastic hinge zones and joints shall not be smaller than #4 bars and spaced no greater than 6 inches on center along longitudinal reinforcement and 12 inches on center along the transverse direction. Cross ties shall directly engage the perimeter longitudinal bars and the location of the 135 degree hook shall be alternated at each tie along the longitudinal bar direction. Cross ties shall be considered adequate for confinement of continuous walls or slabs, hoops shall be used adjacent to areas of high local demand. Special consideration is to be given to locations where these elements experience high axial loads, net tension, or areas at discontinuities and openings. Reinforcement splices, development lengths and details shall be based on ACI 318 or AASHTO LRFD using the appropriate requirements according to the strain and ductility demands determined by analysis.

Commentary: The detailing of an underground box for inelastic behavior is not well defined in current codes, and therefore some interpretation is required to meet the intent of the ACI or Caltrans SDC for this type of structure. The tie spacing and size indicated above are not intended to supersede code requirements which are likely to be more stringent. The designer may look to the requirements in ACI for special concrete moment frames, especially one-sided roof beam to column joints, and in Caltrans for Pier walls loaded out of plane or bridge knee joints. For additional information the designer may also refer to "Caltrans Memo To Designers 6-5" for recommendations of detailing lightly loaded pier walls for inelastic ductility. Also see requirements in Criteria Sections 5.4 and 5.4.12.2. It is desirable to place temperature reinforcement towards the exterior faces of the wall to aid in restraining primary reinforcement buckling and limit cover spalling, however this must be balanced with constructability issues.

3B9.0 VERTICAL SHAFT STRUCTURES

The primary seismic considerations for the design of vertical shaft structures are the curvature strains and shear forces of the lining resulting from ground shear strains due to vertically propagating shear waves. Force and deformation demands are particularly critical in cases where shafts are embedded in deep, soft deposits or cross boundary between two geological strata with stark contrast in stiffness. The general procedure used for evaluating the effects of ground shear strains on shaft structures due to the vertically propagating shear waves is summarized below:

1. Establish the free-field soil/rock shear deformation profile similar to the one shown in Figure 3B-~~8~~**9**, for both MDE and ODE. This shear deformation profile is the result of ground shear strains due to shear waves propagating vertically from the base rock (or very firm base stratum) to the ground surface and shall be estimated by performing free-field site-specific site response analyses using computer program such as SHAKE 91 (Idriss and Sun, 1992). The analyses should account for the various stiffness and damping values (strain dependent) of the various soil and rock layers at the shaft site (refer to discussions in B7.1.1). The analyses should extend from the base firm stratum or the bottom of the shaft, whichever is deeper, to the ground surface.

2. Derive the non-linear springs along the vertical alignment of the shaft structure to represent the varying ground stiffness and strength to be used in the subsequent soil-structure interaction analysis. The non-linear springs should be derived by using the strain-compatible shear modulus obtained from the site-response analyses and considering the diameter/width of the shaft, as well as the discretization of the shaft structure in the structural analysis model.

3. Develop structural models based on the properties and geometry of the shaft structures.

4. The relative lateral shear deformation profile (derived from Step 1 above) between the top (usually the ground surface) and the bottom of the shaft is then applied to the shaft structure model (from Step 3) through the use of equivalent soil/rock springs (from Step 2) to account for the soil-structure interaction effect.

In the analysis, the relative shear deformation is used as the prescribed displacement at the support end of each soil/rock spring.

5. The seismic demands in terms of internal forces (e.g., shear and bending forces) and material strains are computed from the analysis (Step 4) and shall then be combined with non-seismic loads for design and evaluation purposes

3B10.0 LATERAL LOADING FROM NEW OR EXISTING BUILDINGS

Where direct interaction between surface buildings and underground structures occurs, the effects of surface buildings on underground structures, expressed in terms of base shears and/or rocking moments, shall be added to the ground deformation effects on underground structures.

In cases where buildings and underground structures are separated by earth materials, the additional lateral earth pressure due to the inertial forces transmitted from the building through the earth to the underground structures shall be determined and added to the ground deformation effects on the underground structures.

3B11.0 RETAINING WALLS AND U-SECTIONS

For conventional reinforced concrete retaining walls and U-walls, seismic loads expressed in terms of dynamic earth pressures, as outlined in NCHRP Report No. 611, *"Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankment"*, (2008 Transportation Research Board), shall be followed. Special considerations shall be directed to the yielding/non-yielding nature of the walls in determining the dynamic earth pressures. For retaining walls that are allowed to accommodate some limited deformations, depending on their functioning requirements during MDE and ODE, the dynamic earth pressures may be reduced by selecting a design seismic coefficient lower than the peak ground acceleration value (expressed in terms of percent gravity, g). For U-walls, permanent sliding displacement is not likely to occur and therefore the dynamic earth pressures shall be derived based on the non-reduced peak ground acceleration value or the numerical modeling approach similar to that as presented in Section 3B8.1.2.

3B12.0 EFFECTS OF FAULT RUPTURE

The general design philosophy for a tunnel crossing seismically active faults is to accept and accommodate the displacements by either employing an oversized excavation, if appropriate, backfilled with compressible/collapsible material, or using ductile lining to minimize the instability potential of the lining. In cases where the magnitude of the fault displacement is limited or the width of the sheared fault zone is considerable such that the displacement is dissipated gradually over a distance, design of a strong lining to resist the displacement may be technically feasible. The structures, however, will be subject to large axial, shear and bending forces. The analysis and design must consider many important factors including, but nor limited to, the stiffness of the lining and the ground, the angle of the fault plane intersecting the tunnel, the width of the fault, and the magnitude and orientation of the fault movement.

Analytical procedures generally used for evaluating the effects of fault displacement on lining response include structural finite-element and ground spring model and continuum soil-structure finite-element or finite-difference methods. The general procedure is summarized as follows:

1. Characterize the free-field fault displacements (i.e., displacements in the absence of the tunnel) where the fault zone crosses the tunnel, per procedure outlined in Section 2.4 (- Surface Fault Rupture Displacement) of the Metro SSDC.

2. Characterize the soil or rock behavior and derive the corresponding parameters along the longitudinal alignment of the tunnel structure to represent the varying non-linear ground stiffness and strength of the surrounding ground within as well outside the fault zone area. If the structural finite-element and ground spring model is considered appropriate and used in the analysis, then develop the nonlinear transverse and axial (frictional) ground springs to be connected to the tunnel (to model soil normal pressures on the tunnel lining or walls and axial frictional resistance along the tunnel alignment (Figures 3B-~~13~~ **14** and 3B-~~14~~**15**). If the continuum soil-structure finite-element or finite-difference methods are adopted, then develop proper constitutive material laws and corresponding parameters for the surrounding ground to be incorporated in the continuum soil-structure finite-element or finite-difference models.

3. Develop a structural model based on the properties and geometry of the tunnel structure. The non-linear inelastic characteristics of tunnel lining (including the presence of joints and potential hinges) may play an important role in the longitudinal seismic response of a tunnel and hence should be considered in the structural model if applicable.

4. The free-field fault displacement distribution along the length of the tunnel in the fault crossing area (derived from Step 1 above) is then applied to the tunnel structure model (from Step 3) through the use of the non-linear ground springs (from Step 2) in the structural finite-element and ground spring model to account for the ground-structure interaction effect (Figure 3B-~~13~~**14**). If the continuum soil-structure finite-element or finite-difference methods are adopted, then the free-field fault displacement distribution is imposed to the tunnel structure in the continuum soil-structure model through appropriate boundary conditions in the model.

5. The seismic demands in terms of deformations and internal forces computed from the analysis (Step 4) shall then be checked against the capacity of the tunnel structure.

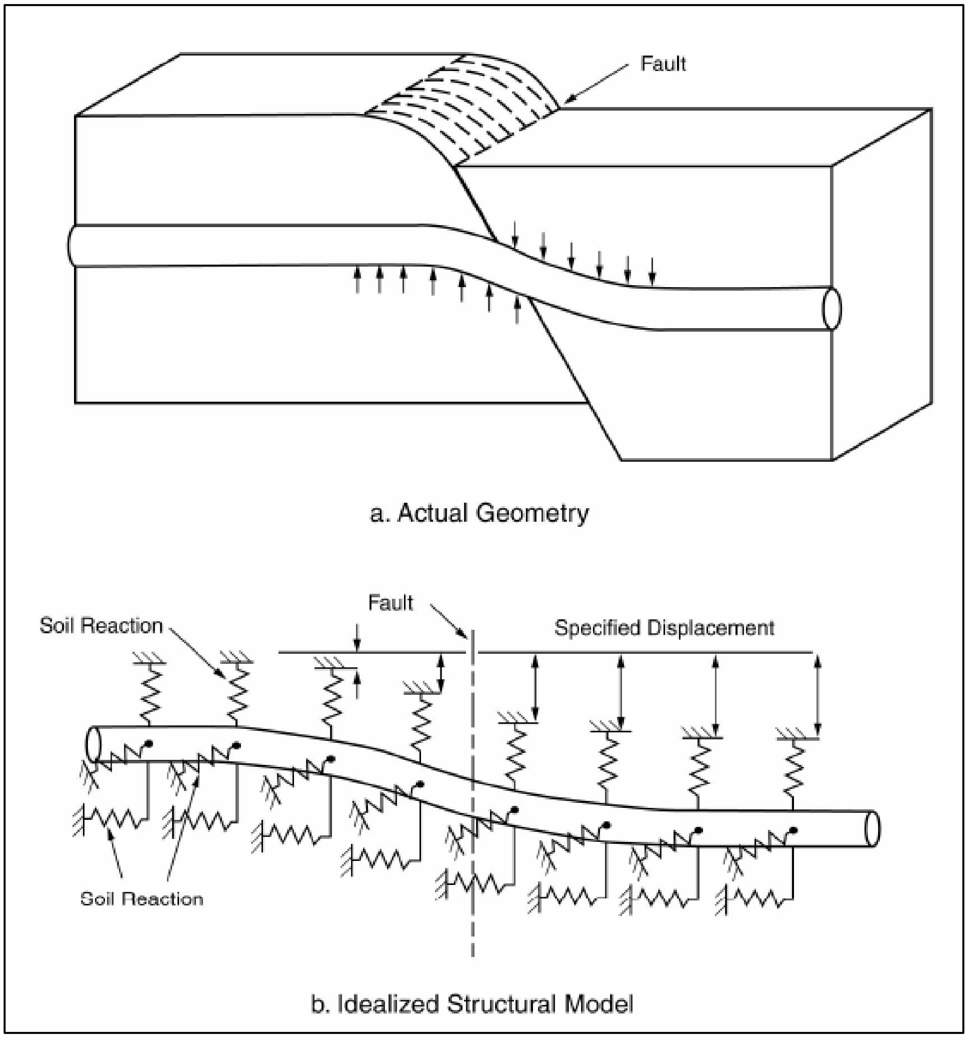


Figure 3B-~~13~~ 14 Tunnel-Ground Interaction Model at Fault Crossing (ASCE Committee on Gas and Liquid Fuel Lifelines, 1984)

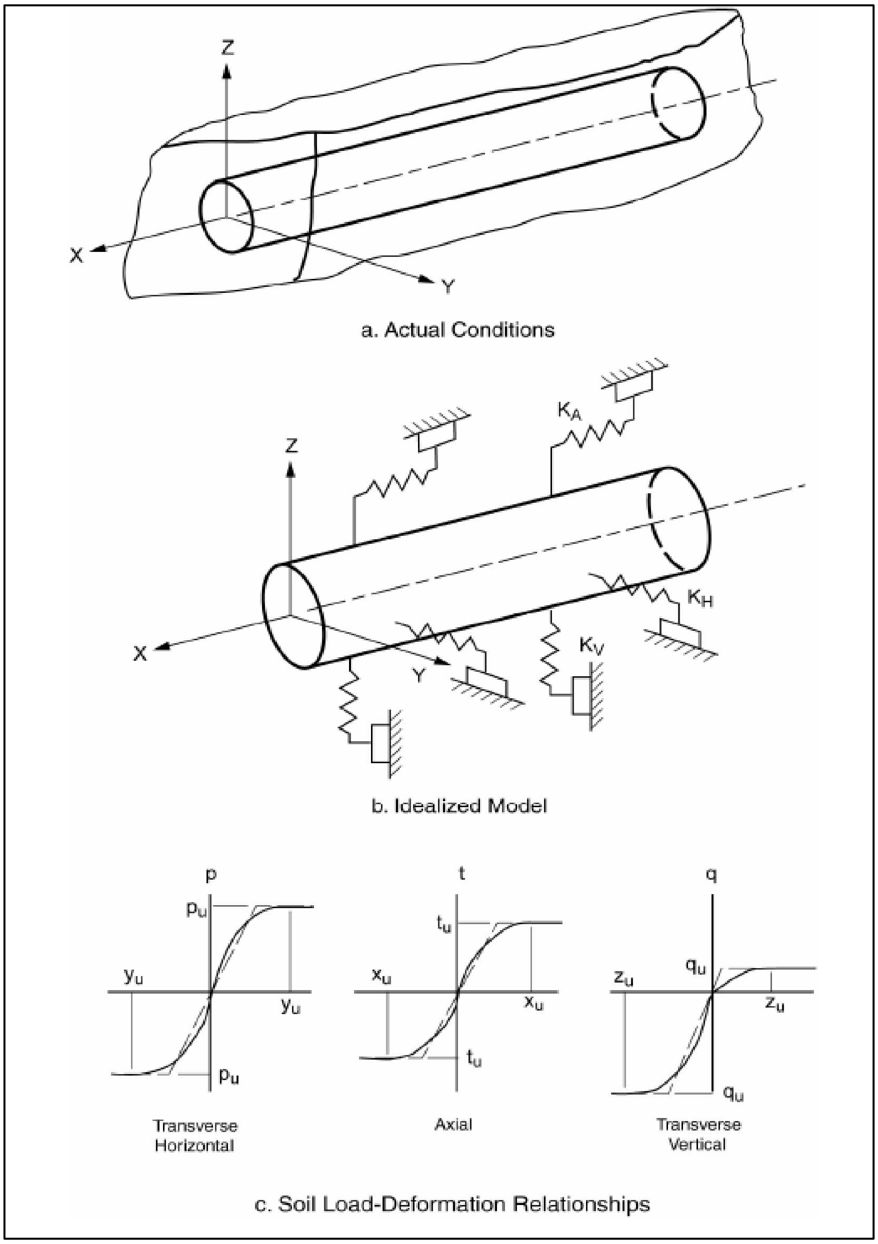


Figure 3B-~~14~~ 15 Analytical Model of Ground Restraint for Tunnel at Fault Crossing (ASCE Committee on Gas and Liquid Fuel Lifelines, 1984)

3B13.0~~SEISMIC DESIGN FOR~~ EFFECTS OF GROUND INSTABILITY

The effects of seismically induced ground instability and permanent deformation of sloping ground or embankments on underground structures shall be considered in the design. For evaluations of slope and embankment stability, including the resulting permanent ground deformations, apply the National Cooperative Highway Research Program Report 611, Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments, latest edition.

3B13.1 General

The stability of the ground surrounding the underground structures along the alignment shall be considered in the design. The surrounding ground includes natural and backfill earth mass located with a zone that may influence the performance of underground structures during and after earthquakes. Ground instability as a result of seismic shaking can include liquefaction, post-liquefaction settlements, and slope/embankment instability (landslide)

3B13.2 Effects of Liquefaction and Permanent Ground Deformations

The effects of liquefaction and liquefaction-induced ground deformations shall be evaluated at relevant locations along the project alignment including tunnels, the shafts, the stations, and potential slope instability affecting the structures. These shall include the following:

- Uplift, buoyancy, and flotation of the tunnels, stations, and other underground structures;
- Post-liquefaction settlements and deformations (total as well as differential;
- Lateral sliding stability of the tunnels and other underground structures;
- Loss of bearing capacity, if applicable;
- Down-drag and reduction in lateral/vertical resistance of deep foundations supporting the underground structure, if applicable.

For soil layers in which the safety margin against initial liquefaction (triggering) is unsatisfactory, a liquefaction impact analysis based primarily on a deformation approach shall be performed. Potential impacts of liquefaction include tunnel floatation, uplift pressure, increased lateral earth pressure, down-drag force, bearing capacity failure, loss of lateral support (for piles or other deep foundations), lateral spread and slope stability problems, and post-liquefaction settlements and differential settlements. Relatively dense soils that liquefy may subsequently harden or stabilize at small deformations (cyclic mobility) and thus have relatively small impact on structures. Conversely, relatively loose soils that liquefy tend to result in much larger post-liquefaction deformations.

For underground structures, the depth of liquefaction investigation shall extend to a depth that is a minimum of 80 feet below the existing ground surface of final grade, whichever is deeper.

The proposed structures shall be designed to accommodate not only the total ground deformations but also the differential deformations. The minimum differential ground settlements to be used in the design shall be one-half of the total settlement at sites where natural soils underlie the structures. When the subsurface condition varies significantly in lateral directions and/or the soils are of Holocene deposits and/or artificial fills a minimum value of greater than one-half of the total settlement shall be used as the differential settlements.

The maximum deformation due to the differential tunnel movement (combined non-seismic and seismic movements) shall not cause long tem leakage of the tunnel structures, including at its interface connections to other structures.

3B13.3 Effects of Landslide and Slope Stability

The potential for seismically induced landslides and slope instability shall be identified along the proposed alignment. If quasi-static seismic stability analysis is performed for permanent structure, the seismic coefficient shall be determined in accordance with the NCHRP Reports, "Seismic Analysis and Design of Retaining Walls, Buried Structures, slopes, and Embankment, (TRB 2008, NCHRP 2008).

For quasi-static slope stability analysis, the factor of safety shall not be less that 1.1. If the computed factor of safety is less than 1.1, an impact study shall be performed based on earthquake-induced slope movements, using a refined and more accurate method of analysis such as the Newmark Time-History Analysis or dynamic non-linear soil continuum method of analysis to estimate the movements. The Newmark Time-History Analysis is described in the National Cooperative Highway Research Program (NCHRP) Reports (611 and Volume 2 on Project 12-70), "Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments". The impact of the potential slope movements on the affected structures shall be assessed. If the impact assessments yield unacceptable performance of the structures, appropriate mitigation measures shall be incorporated into the design.

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END OF SECTION 5 APPENDIX
METRO SUPPLEMENTAL SEISMIC DESIGN CRITERIA