

Caltrans/Division of Structures

San Francisco-Oakland Bay Bridge - Design Criteria

Contract 59A0040

**100% Submittal**

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**San Francisco-Oakland Bay Bridge  
East Span Seismic Safety Project**

**Contract 59A0040**

***SELF-ANCHORED SUSPENSION BRIDGE***

***DESIGN CRITERIA***

**07/15/02**

**100% Submittal**

Reference: Criter20r22

Prepared by T.Y.Lin International/Moffatt & Nichol Engineers, a Joint Venture

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1. GENERAL

The bridge shall be designed in accordance with "Caltrans Bridge Design Specifications Manual (1995) (BDS)," modified or augmented as detailed in this design criteria.

In addition to bridge and site specific criteria, pertinent sections of the following standards or codes have been employed for such modifications or augmentations.

- "AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges", 1999 Edition, with Addenda through 1998 "AASHTO Seg."
- "Proposed LRFD Guide Specifications for Design of Segmental Concrete Bridges", S. I. Units, March 1997 "LRFD Seg."
- "AASHTO Standard Specifications for Highway Bridges", 16<sup>th</sup> Edition, 1992 "AASHTO"
- "AASHTO LRFD Specifications for Highway Bridge Design" 2<sup>nd</sup> Edition, 1998, AASHTO, Washington, DC, 1994 "AASHTO LRFD"
- "Sacramento Regional Transit District Light Rail Design Criteria", May 1993 "Sac. LRT"
- "San Francisco-Oakland Bay Bridge East Span Seismic Safety Project Light Rail Transit Design Criteria", 1999 "SFOBB LRT"
- "Improved Seismic Design Criteria for California Bridges: Provisional Recommendations" ATC-32 Report, June 30, 1996 "ATC-32" Report
- "Proposed Design Specifications for Steel Box Girder Bridges", FHWA-TS-80-205, FHWA, Washington, DC, 1980
- "Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges" Volume I: Final Report, February, 1991
- "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Working Stress Design", API RP2A-WSD 20<sup>th</sup> Ed. 1993
- "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design" – API RP2A-LRFD 1<sup>st</sup> Ed. 1993
- "AISC Manual of Steel Construction Load & Resistance Factor Design" (LRFD), 1999 Edition "AISC LRFD"
- "AISC Manual of Steel Construction", 9<sup>th</sup> Edition, 1989 "AISC"
- "ANSI/ASCE", 7-95 Standard "ANSI / ASCE"
- Stability Design Criteria, 5<sup>th</sup> Edition, SSRC
- West Wind Laboratory Report, Monterey, California, May, 2001 "WWL Report"

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| • Transportation and Traffic Engineering Handbook, Institute of Transportation Engineers, 1976   | "THAT"   |
| • Technical Specifications for Suspension Structures   | "Suspension Contract" Document                               |
| • Hydraulic Modeling and Scour Analysis, T.Y. Lin / Moffatt & Nichol, JV Report, October 31, 1999  | "Scour" Report   |
| • Seismic Ground Motion for SFOBB, East Span Seismic Safety Project, Furgo, Earth Mechanics, JV Report, February 2001  | "Seismic Ground Motion" Report                               |
| • AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridge, 1991   | "AASHTO Vessel Collision"                                    |
| • Bridge Welding Code, AWS D1.5M, 1996   | "AWS D1.5"   |
| • Structural Welding Code – Steel AWS D1.1, 1998   | "AWS D1.1"   |
| • AISC – Seismic Provisions for Structural Steel Buildings, April 1997   | "AISC – Seismic Provisions"                                  |
| • "The strength and ductility of steel bridge piers based on loading tests" by K. Kawashima, G. MacCrae, and K. Hasegawa, Public Works Record Institute, Journal of Research Vol. 29, March 1992                         | "Kawashima et al" Report                                     |
| • Technical Specifications for Skyway Sidewalks  |  |
| • "Recommended Design Loads for Bridges" by the Committee on Loads and Forces on Bridges of the Committee on Bridges of the Structural Division, July 1981   | "Design Loads for Bridge" Report                             |
| • "East Bay Bridge Vessel Collision Analysis", Moffatt & Nichol  |  |
| • "Frictional Resistance Between Cable and Saddle Equipped with Frictional Plates" by K. Hasegawa, H. Kojima, M. Sasaki, and K. Takena, Journal of Structural Engineering, Vol. 121, No. 1, January 1995, Paper No. 4042 | "Vessel Collision Analysis" Report<br>"Hasegawa et al" Paper |
| • "Slip Behavior of Cable Against Saddle Suspension Bridges" by K. Takena, M. Sasaki, K. Hata, and K. Hasegawa. Journal of Structural Engineering, Vol. 118, No. 2, February 1992, Paper No. 1309                        | "Takena et al" Paper   |
| • SFOBB East Span; Seismic Design Criteria Basis, SDCB, June 27, 2000  | "SDCB"   |
| • Axial Pile and Drivability Main Span-East Pier and Skyway Structures, Fugro and Earth Mechanics Report, March 2001   | "Axial Pile and Drivability" Report                          |
| • Lateral Pile Design for Main Span Pier E2 and Skyway, Fugro and Earth Mechanics Report, February 2001  | "Lateral Pile Design" Report                                 |
| • San Francisco Oakland Bay Bridge –New Self Anchored Suspension Span Wind Studies Final Report, January 2002  |  |

## 2. DESIGN LOADS

This section covers all design loads except for seismic demands discussed in Section 7. Unless specified herein or in BDS, all loads shall be as specified in the standards and codes cited in Section 1 of this criteria.

### 2.1 STRUCTURAL DEAD LOADS – DL

#### 2.1.1 Concrete

The in-service air dry unit mass of normal weight concrete, including reinforcement shall be 2480 kg/m<sup>3</sup> (155 lb/ft<sup>3</sup>).

The in-service air dry unit mass of sand lightweight concrete including reinforcement shall be 1920 kg/m<sup>3</sup> (120 lb/ft<sup>3</sup>).

#### 2.1.2 Steel

The unit mass of structural steel, including fabricated plate steel, rolled shapes, and wire, shall be 7850 kg/m<sup>3</sup> (490 lb/ft<sup>3</sup>).

### 2.2 OTHER PERMANENT LOADS – SDL

Other permanent loads are assumed to be applied at the time of construction except for utilities and dead load of Light Rail Transit (LRT) appurtenances.

#### 2.2.1 Vehicular and Pedestrian Barriers

Concrete roadway barrier	
Modified 732 barrier	8.5 kN/m (585 lb/ft)
Steel barrier	3.0 kN/m (206 lb/ft)
Safety rail	0.285 kN/m (20 lb/ft)
Steel pedestrian handrail	0.60 kN/m (41 lb/ft)

#### 2.2.2 Wearing Surface

Suspension structure – 50 mm (2 in)	1.16 kN/m <sup>2</sup> (24 lb/ft <sup>2</sup> )
Bicycle/pedestrian path – 13 mm (1/2 in)	0.31 kN/m <sup>2</sup> (6.5 lb/ft <sup>2</sup> )

#### 2.2.3 Maintenance Travelers and Support System

Traveler supporting system (SDL <sub>MT</sub> )	
a. Eastbound structure	3.65 kN/m (250 lb/ft)
b. Westbound structure	3.65 kN/m (250 lb/ft)

**2.2.4 Provision for Utilities**

Eastbound and Westbound structures 12.6 kN/m (869 lb/ft)

This loading includes allowance for miscellaneous metal, drainage, lighting system, utilities, utility supports, service platform utilities, and service platform support.

**2.3 LIVE LOADS – LL+I**

Live loads shall be considered for two roadway configurations:

- Highway traffic (6 lanes) on the Westbound Structure; highway traffic (6 lanes) plus a bicycle/pedestrian facility on the south side of the Eastbound Structure.
- An LRT system on the inside lane plus highway traffic (4 lanes) on the Westbound Structure; an LRT system on the inside lane plus highway traffic (4 lanes) on the Eastbound Structure with a bicycle/pedestrian facility on the south side.
- All design live loads shall be positioned both transversely and longitudinally so as to produce the maximum influence on the structure.

**2.3.1 Standard Truck and Lane Loads**

Standard vehicular live load shall be AASHTO HS20-44 with modified lane loads for the suspended spans as follows:

Loaded Length L (m) (L (ft))	Uniform Load kN/m/lane (lb/ft/lane)	Concentrated Load kN/lane (lb/lane)
0 < L ≤ 183 (0 < L ≤ 600)	9.34 (640)	115.7 (26,000)
183 < L < 366 (600 < L ≤ 1200)	11.68-L/78.3 (800-L/3.75)	144.6-L/6.33 (32,500-10.833L)
366 ≤ L (1200 ≤ L)	7.01 (480)	86.7 (19,500)

"Design Loads for Bridge"  
Report

In the design of the orthotropic deck, one wheel load of 71 kN (16 kips) or two wheel loads of 54 kN (12 kips) each spaced 1.2 m (4 ft) apart, whichever produces the greater stress, shall be used.

**2.3.2 Load Reduction Factors for Multiple Lane Loading**

Load reduction factors for multiple lane loading shall be applied to highway lanes only. Full LRT Train Loading shall be combined with highway traffic loading to produce the most critical stress condition in the member.

**2.3.3 Permit Vehicle Loads**

Permit loads (P13) shall be considered for design of the bridge superstructure, and the orthotropic deck.

BDS 3.7.7, Figure 3.7.7A,  
 BDS Figure 3.7.7B and  
 3.11.4.3

**2.3.4 Pedestrian Loads**

Pedestrian loading shall be in accordance with BDS.

BDS 3.14.1

**2.3.5 Maintenance Vehicle**

For the bike path, one pick-up truck, including impact shall be used in the deck design.

"THAT", Table 2.3

Pick-up truck (gross weight)                      40 kN (8900 lb)

**2.3.6 Live Load Contribution Under Seismic Conditions**

A reduced live load shall be considered for combination with seismic demands (this group load will be labeled Group VIIA loading). The reduced loading factor  $\beta_{LE}$  shall represent estimated peak hour traffic predictions for the year 2025.

**2.3.7 Live Load on Platforms and Landings**

**2.3.7.1 General**

Minimum    4.0 kN/m<sup>2</sup> (85 psf)

**2.3.7.2 Tower and Elevator Access and Landings**

VAB Machine    4.45 kN (1000 lb)

**2.3.7.3 Utility Platform**

For live load on utility platform, see Mechanical Design Criteria

**2.3.8 Maintenance Traveler**

**2.3.8.1 Live Load on Traveler**

The traveler shall be designed for the following loads:

Live load (LL<sub>MT</sub>)

- a. Area less than 28 m<sup>2</sup> (300 ft<sup>2</sup>)                      1.25 kN/m<sup>2</sup> (25 lb/ft<sup>2</sup>)
- b. Area more than 28 m<sup>2</sup> (300 ft<sup>2</sup>)                      0.75 kN/m<sup>2</sup> (15 lb/ft<sup>2</sup>)
- c. Concentrated load in addition to a, b                      4.4 kN (1000 lb)

Impact (I):    10% (DL<sub>MT</sub> + LL<sub>MT</sub>)

Load in direction of travel:  
 Maximum of wind, or 20% (DL<sub>MT</sub> + LL<sub>MT</sub>)

**2.3.8.2 Live load on SAS Bridge**

The SAS Bridge shall be designed to carry the gross weight (dead load plus live load of the traveler) as live load.

Total weight of traveler with platform ( $DL_{MT}$ ):

- a. Eastbound structure 414 kN (93,000 lb)
- b. Westbound structure 370 kN (83,100 lb)

**2.4 THERMAL EFFECTS – T**

BDS 3.16

**2.4.1 Uniform Temperature –  $T_c$  and  $T_s$**

Design temperature range shall correspond to BDS requirements for coastal areas:

Mean Temperature	20° C (68° F)
Rise or Fall	
Concrete	17° C (30° F)
Steel	22° C (40° F)

**2.4.2 Coefficient of Thermal Expansion**

“AASHTO Seg.” 6.4.3

Normal Weight Concrete	10.8x10 <sup>-6</sup> /°C	(6.0x10 <sup>-6</sup> /°F)
Steel	11.7x10 <sup>-6</sup> /°C	(6.5x10 <sup>-6</sup> /°F)

**2.4.3 Concrete Temperature Gradient –  $DT_c$**

“AASHTO Seg.” 6.4.4,  
 Figure 6.4

Positive and negative temperature gradients, diagrammed on Fig. 6.4 of the “AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges”, shall correspond to the values listed below.

<u>Positive Gradient</u>		<u>Negative Gradient</u>	
T <sub>1</sub> =	30.0° C (54° F)	T <sub>1</sub> =	-15.0° C (-27° F)
T <sub>2</sub> =	7.8° C (14° F)	T <sub>2</sub> =	-3.9° C (-7° F)
T <sub>3</sub> =	0° C (0° F)	T <sub>3</sub> =	0° C (0° F)
A =	300 mm (12 in)	A =	300 mm (12 in)

**2.4.4 Steel Temperature Gradient –  $DT_s$**

A temperature gradient of 11° C (20° F) between the top and bottom of the box will be considered for global stresses in the design of the orthotropic deck.

Negative temperature gradients need not be considered.

**2.5 STREAM FLOW – SF**

BDS 3.18

Forces due to stream flow shall be applied to each substructure in accordance with BDS, using the 100-year return period pool elevation and velocities from the “Hydraulic Modeling and Scour Analysis” report.

“Scour” Report

2.6 WIND - W

2.6.1 Main Span Suspension Bridge

The Self-Anchored Suspension (SAS) bridge shall be designed for wind-induced aerodynamic loading based on a site-specific wind hazard assessment. The design shall be based on: (a) service load analyses - using aerodynamic coefficients obtained by wind tunnel testing - to determine "equivalent static" load demands on structural members, (b) aeroelastic analyses to assess the dynamic stability of the structure against torsional divergence and flutter instability - using aeroelastic flutter derivatives obtained from wind tunnel testing, and (c) vortex shedding response limited to acceptable acceleration and displacement thresholds.

The design shall be based on the ANSI/ASCE 7-95 Standard and the wind hazard study as well as the wind tunnel testing to be performed by West Wind Laboratory.

*Service Load Analysis*

For the service load analyses of the SAS bridge, equivalent static wind loading on the deck structure as well as the main tower and piers shall be computed. The aerodynamically induced distributed loads due to lift, drag, and pitching moment (torsion) shall be computed as follows:

$$\begin{aligned} F_{LIFT} &= q G C_L B && \text{(lift force per unit length)} \\ F_{DRAG} &= q G C_D B && \text{(drag force per unit length)} \\ M_{PITCH} &= q G C_M B^2 && \text{(torsional moment per unit length)} \end{aligned}$$

where:

- q = dynamic pressure  
=  $\frac{1}{2} \rho U^2$
- $\rho$  = mass density of atmospheric air
- U = design wind speed
- G = Gust effect factor (to be determined as per ANSI / ASCE 7-95)

$C_L, C_D, C_M$  = aerodynamic coefficients of Lift, Drag, and Pitching Moment (torsion), respectively, per unit length of span, obtained from wind tunnel testing

B = along-wind characteristic dimension (e.g., span width) used in the measurement of the aerodynamic coefficients

The design wind speed, U, shall be computed based on the following two step procedure:

BDS 3.15

"ANSI / ASCE" 7.95

"WWL Report"

**Reference Wind Speed**

Obtain structural design reference wind speed,  $U_{REF}$ , based on the site-specific wind hazard study, based on extreme wind return period and mean value of annual extreme wind. The design reference speed is given as:

$$U_{REF} = 137 \text{ km/h (85 mph)}$$

$U_{REF}$  is for:

$T = 3$  s averaging time

$z_1 = 10$  m reference elevation

$z_{01}$  = reference surface roughness (0.07 m for open exposure – Exposure C)

$N = 50$  years return period

**Design Wind Speed**

The design wind speed is then calculated after adjusting for the averaging time, elevation, return period, and exposure. The design wind speed is:

$$U_{DESIGN} = 148 \text{ km/h (92.1 mph)}$$

$U_{DESIGN}$  is for:

$T = 1$  hour averaging time

$z_1 = 50$  m elevation

$z_{01} =$  Exposure D (Coastal)

$N = 100$  year return period

The following velocity profile may be used to evaluate the wind speed at various heights:

$$U(z, z_0) = 2.5 U^* \ln(z / z_0); \quad z = \text{elevation above datum line}$$

The requirements for wind tunnel testing and aeroelastic analyses are specified in Section 3.2.

**2.6.2 E2 – Hinge A Transition Span**

The E2 – Hinge A transition span structure shall be designed for wind loads in accordance with Caltrans BDS and the ANSI/ASCE 7-95 Standard.

The reference wind speed shall be based on a 100-year return period for compatibility with the design of the cable-supported portion of the SAS bridge. Nominal loads and/or stresses and resistance factors specified in the BDS shall be met by all bridge components.

“WWL Report”

**2.7 SHIP COLLISION – SC**

“AASHTO Guide Specifications and Commentary for Vessel Collision Design,” Volume I, Final Report, February 1991 shall be used to determine the ship impact force.

The bridge is classified as “critical bridge” and the acceptable annual frequency of collapse shall be equal to or less than 0.01 in 100 years. The design vessel shall be selected based on Method II.

“AASHTO Vessel Collision”

“Vessel Collision Analysis” Report

**2.8 COMBINATION OF LOADS**

**2.8.1 Service and Ultimate Limit States**

Loading combinations shall be in accordance with Caltrans BDS Tables 3.22.1A and 3.22.1B, “AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges” and “Sacramento Regional Transit District Light Rail Design Criteria”.

Tables 2.8.1A and 2.8.1B show the applicable load combinations for Service Load Design and Load Factor Design, respectively.

Dead Load – DL – shall include structural dead loads (DL), other permanent loads (SDL) and erection loads (EL) where applicable.

Permanent effects of creep and shrinkage shall be added to all Service Load Design combinations with a load factor of 1.0.

When checking tensile stresses for Service Load Design the variable load effects shall be divided by the allowable stress increases.

One half of the temperature gradient shall be used for load combinations that include full live load plus impact.

“AASHTO Seg.” 7.0

“AASHTO Seg.” 7.2

Group Loading Combinations for Service Load Design and Load Factor Design are given by:

$$\text{Group (N)} = \gamma \{ [\beta_D D + \beta_L(L+I) + \beta_C CF + \beta_E E + \beta_B B + \beta_{SF} SF + \beta_W W + \beta_{PS} PS + \beta_{WL} WL + \beta_{LF} LF + \beta_R(R+S+T+DT) + \beta_{EQ} EQ] + \alpha_{LRT} [\beta_{L,LRT}(L_{LRT}+I_{LRT}) + \beta_{CF,LRT} CF_{LRT} + \beta_{NF,LRT} NF_{LRT} + \beta_{LF,LRT} LF_{LRT} + \beta_{WL,LRT} WL_{LRT}] \}$$

where :

N	Group No.
$\gamma$	Load Factor, see Tables 2.8.1A and 2.8.1B
$\beta$	Coefficient, see Tables 2.8.1A and 2.8.1B
D	Dead Load
L	Live Load
I	Live Load Impact
E	Earth Pressure
B	Buoyancy
W	Wind Load on Structure
WL	Wind on Live Load
LF	Longitudinal Force from Live Load
CF	Centrifugal Force
R	Rib Shortening
S	Shrinkage
T	Temperature
DT	Thermal Gradient
EQ	Earthquake
SF	Stream Flow Pressure
PS	Prestress
$\alpha_{LRT}$	1 or 0, corresponding to presence or absence of LRT Train
$L_{LRT}$	LRT Live Load
$I_{LRT}$	Impact from LRT Live Load
$CF_{LRT}$	Centrifugal Force from LRT Train
$NF_{LRT}$	Horizontal Force from LRT Train
$LF_{LRT}$	Longitudinal Force from LRT Train
$WL_{LRT}$	Wind on LRT Train

**2.8.2 Additional Thermal Loading Combination – Concrete**

In addition to Service Load Design Combinations defined in Table 2.8.1A, the following load combination shall apply:

$$(DL + SDL + EL) + \beta_E E + B + SF + R + S + T + DT$$

where :

DL	Dead Load
SDL	Superimposed Dead Load
EL	Erection Load in the final state

100% allowable stress applies to this load combination.

**2.8.3 Construction Considerations – Concrete**

Allowable tensile stresses for construction load combinations shall be checked in accordance with "AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges", 1999.

"AASHTO" 7.2.2

"AASHTO" 7.4,  
Table 7.2

**2.8.4 Construction Considerations – Steel**

Where steel units are assumed to be erected by balanced cantilever construction, the provisions of Section 2.8.4 shall apply.

Where steel units are assumed to be erected by conventional crane picks, floating cranes, heavy lifts, and float-ins, their adequacy may be investigated including dynamic load and impact factors using:

- Service Load Design at 125% of the basic allowable stresses for the final condition
- Load Factor Design

**2.8.5 Ship Impact Load Combination**

The vessel collision impact forces are combined with other loads. The group loading combination has the same format as that used in the current BDS for seismic design (Group VII) with all gamma and beta factors equal to 1.0 and the ship impact force replacing the earthquake load.

The ship impact load combination is given by the following expression:  
Group Load =  $\gamma (1.0 D + 1.0 P + 1.0 B + 1.0 SF + 1.0 E)$

where :

$\gamma$	1.0 for all design methods
D	Dead Load
P	Vessel Collision Impact Force
B	Buoyancy
SF	Stream Flow Pressure
E	Earth Pressure

**2.8.6 Additional Group Loading Combinations**

In addition to load combinations described elsewhere in this criteria, the following two load combinations shall be considered for Load Factor Design:

$$\text{Group VIIA} = \gamma (1.0 D + \beta_{LE} (L + I)_H + 1.0 E + 1.0 B + 1.0 SF + 1.0 PS + 1.0 EQ)$$

$$\text{Group XIA} = \gamma (1.0 D + 1.0 E + 1.0 B + 1.0 SF + 1.0 PS + 1.0 P)$$

where :

$\gamma$	1.0 for all design methods
$\beta_{LE}$	0.17 (see Section 2.3.6)
D	Dead Load
P	Vessel Collision Impact Force
B	Buoyancy
SF	Stream Flow Pressure
E	Earth Pressure
PS	Prestress
EQ	Earthquake
$(L + I)_H$	Horizontal Live Load (Highway/Bicycle/Pedestrian/LRT)

Table 2.8.1A Service Load Design

Group	Y factor	D	L+I	CF	E	B	SF	W	WL	LF	PS	R+S	T	DT	$\frac{C_{LRT}}{L_{LRT}+I_{LRT}}$	$\frac{C_{LRT}}{CF_{LRT}}$	$\frac{C_{LRT}}{NF_{LRT}}$	$\frac{C_{LRT}}{LF_{LRT}}$	$\frac{C_{LRT}}{(WL_{LRT})}$	% Allow.
I	1.0	1	1	1	1	1	1	0	0	0	1	1	0	0	1	1	0	0	0	100
I <sub>LRT</sub>	1.0	1	0	0	1	1	1	0	0	0	1	1	0	0	1	1	0	0	0	100
II	1.0	1	0	0	1	1	1	1	0	0	1	1	0	0	0	0	0	0	0	125
III	1.0	1	1	1	1	1	1	0.3	1	1	1	1	0	0	1	1	1	1	1	125
IV	1.0	1	1	1	1	1	1	0	0	0	1	1	1	0.5	1	1	1	0	0	125
V	1.0	1	0	0	1	1	1	1	0	0	1	1	1	1	0	0	0	0	0	140
VI	1.0	1	1	1	1	1	1	0.3	1	1	1	1	1	0.5	1	1	1	1	1	140
VIA*	1.0	1	0	0	1	1	1	0	0	0	1	1	0	1	0	0	0	0	0	100

AASHTO Seg 8.2.2

This portion of the Table only applies when LRT is considered.

where:

- D Dead Load
- L Highway + Bicycle/Pedestrian Live Load
- I Highway + Bicycle/Pedestrian Live Load Impact
- E Earth Pressure
- W Wind Load on Structure
- WL Wind on Live Load
- LF Longitudinal Force from Live Load
- CF Centrifugal Force
- R Rib Shortening
- S Shrinkage
- T Temperature
- DT Thermal Gradient
- SF Stream Flow Pressure
- PS Prestress

1 or 0 corresponding to presence or absence of LRT Train

- $C_{LRT}$  LRT Live Load
- $L_{LRT}$  Impact from LRT Live Load
- $I_{LRT}$  Centrifugal Force from LRT Train
- $CF_{LRT}$  Longitudinal Force, LRT Train
- $LF_{LRT}$  Nosing Force, LRT Train
- $NF_{LRT}$  Wind on LRT Train
- $WL_{LRT}$

\* ASBI 8.2.2: Factors for erection loads (EL) at the final state of completion are not included in this Table. SFRAME analysis automatically accounts for these loads.

Table 2.8.1B Load Factor Design

Group	$\gamma$ factor	D	(L+I)H	(L+I)P	CF	E	B	SF	W	WL	LF	PS	R+S	T	DT	EQ	$\alpha_{LRT}$ $L_{LRT}+I_{LRT}$	$\alpha_{LRT}$ $CF_{LRT}$	$\alpha_{LRT}$ $NF_{LRT}$	$\alpha_{LRT}$ $LF_{LRT}$	$\alpha_{LRT}$ $WL_{LRT}$	RT <sub>2</sub> of BR	LR <sub>2</sub> of BR	$\alpha_{LRT}$ $DF_{LRT}+$ I
I	1.30	$\beta_b$	1.67	0	1	$\beta_E$	1	1	0	0	0	0.77	0	0	0	0	1	1	0	0	0	0	0	0
I <sub>FSD</sub>	1.30	$\beta_b$	1	1.25	1	$\beta_E$	1	1	0	0	0	0.77	0	0	0	0	1	1	1	0	0	0	0	0
II	1.30	$\beta_b$	0	0	0	$\beta_E$	1	1	1	0	0	0.77	0	0	0	0	0	0	0	0	0	0	0	0
III	1.30	$\beta_b$	1	0	1	$\beta_E$	1	1	$\beta_{w1}$	1	1	0.77	0	0	0	0	1	1	1	1	1	0	1	0
IV	1.30	$\beta_b$	1	0	1	$\beta_E$	1	1	0	0	0	0.77	1	1	0.5	0	1	1	0	0	0	1	1	0
V	1.25	$\beta_b$	0	0	0	$\beta_E$	1	1	1	1	0	0.80	1	1	1	0	1	0	0	0	0	1	1	0
VI	1.25	$\beta_b$	1	0	1	$\beta_E$	1	1	$\beta_{w1}$	1	1	0.80	1	1	0.5	0	1	1	1	1	1	1	1	0
VII	1.00	1.0	0	0	0	$\beta_E$	1	1	0	0	0	1	0	0	0	1	0	0	0	0	0	0	0	0
XI	1.30	$\beta_b$	1	0	1	$\beta_E$	1	1	0	0	0	0.77	0	0	0	0	0	1	0	0	0	0	0	1

This portion of the Table only applies when LRT is considered

$\beta_b = 0.75$  when checking columns for maximum moment or maximum eccentricities and associated axial load;

and when Dead Load effects are of opposite sign to the net effects of other loads in a Group.

$\beta_b = 1.00$  when checking columns for maximum axial load and associated moment.

$\beta_E = 0.5$  for checking positive moments in rigid frames.

$\beta_E = 1.00$  for vertical earth pressure.

$\beta_E = 1.30$  for lateral earth pressure

$\beta_{w1} = 0.3$  for highway/bicycle/pedestrian loads only

$\beta_{w1} = 1.3$  for LRT+highway/bicycle/pedestrian loads only

where:

- D Dead Load
- (L+I)H Highway + Bicycle/pedestrian Live Load plus Impact
- (L+I)P Permit Load plus Impact
- E Earth Pressure
- B Buoyancy
- W Wind Load on Structure
- WL Wind on Live Load. 1.46 kN/m (100 lb/ft)
- LF Longitudinal Force from Live Load
- CF Centrifugal Force
- R Rib Shortening
- S Shrinkage
- T Temperature
- DT Thermal Gradient
- EQ Earthquake
- SF Stream Flow Pressure
- PS Prestress
- $\alpha_{LRT}$  1 or 0 corresponding to presence or absence of LRT Train
- $L_{LRT}$  LRT Live Load
- $I_{LRT}$  Impact from LRT Live Load
- $CF_{LRT}$  Centrifugal Force from LRT Train
- $NF_{LRT}$  Nosing Force, LRT Train
- $LF_{LRT}$  Longitudinal Force, LRT Train
- $WL_{LRT}$  Wind on LRT Train

For suspension spans, applied loads shall be factored before stiffness analyses are performed. Furthermore,  $\gamma$  x  $\beta_d$  is taken as 1.0 whenever it controls the design.

Linear superposition of element forces and stresses is not valid and shall not be used.

### 3. SPECIAL LOAD REQUIREMENTS FOR SAS BRIDGE

#### 3.1 LIVE LOADS

##### 3.1.1 Lane Load Intensity

Lane loads for live load analysis of the *span* shall be taken from a site-specific loading study. The study shall determine the lane load intensity as a function of loaded length. Pending completion of the study, the table in Section 2.3.1 shall be used.

##### 3.1.2 Cross-Lane Distribution

For load cases with multiple lanes loaded, the individual lane loads may be reduced in intensity by the following cross-lane distribution factors.

Cross-Lane Distribution Factors

Number of Lanes	1	2	3	4	5	6
Factor	1.0	1.0	0.9	0.75	0.75	0.75

##### 3.1.3 Impact

Impact may be neglected for analysis of the *span*.

#### 3.2 WIND LOAD

In order to ensure a stable and safe performance of the SAS bridge under wind-induced aerodynamic effects -- which are known to play a major role on the behavior of cable-supported bridges -- wind-tunnel testing and aeroelastic analyses shall be performed.

The wind tunnel testing and aeroelastic analysis program shall be used to ensure the following:

- a) Aeroelastic flutter instability is prevented. This shall be accomplished by ensuring that the insipient flutter instability wind speed of the deck structure (established by wind tunnel testing and aeroelastic analyses) is well above the critical flutter threshold based on site-specific wind hazard study. The critical flutter threshold speed is established as: 214 km/h (133 mph) for an elevation of 50 m, 10 min averaging time, and 10000 year return period.
- b) Vortex shedding behavior of the deck and towers under prevailing wind conditions is maintained within strict deflection and acceleration limits. Vertical acceleration of the deck shall be limited to 0.02 g for wind speeds under 15 m/s and to 0.05 g for speeds exceeding 15 m/s.
- c) Static torsional divergence is prevented.

BDS 3.12

"WWL Report"

"WWL Report"

- d) All stress levels remain well within allowable stresses.
- e) Cable and hanger vibration is minimized.

### 3.3 FATIGUE

Orthotropic decks, box girders and cross beams shall be designed according to the Load and Resistance Factor Design (LRFD) provisions of AASHTO. Fatigue design shall provide resistance for more than two million stress cycles.

## 4. CONCRETE SUBSTRUCTURE

For cast-in-place construction, use Caltrans Bridge Design Specification Manual (BDS), except as modified below.

### 4.1 ALLOWABLE STRESSES

BDS shall apply, except as modified below.

#### 4.1.1 Concrete

##### 4.1.1.1 Joint and Web Shear

4.1.1.1.1 Principal tensile stress limits for Functional Evaluation Earthquake (FEE), or without special reinforcement:

Normal weight concrete	$0.29 \sqrt{f'_c}$ (MPa)
	$[3.5 \sqrt{f'_c}]$ (psi)

4.1.1.1.2 Principal tensile stress limits for Safety Evaluation Earthquake (SEE):

Normal weight concrete	$0.42 \sqrt{f'_c}$ (MPa)
	$[5 \sqrt{f'_c}]$ (psi)

4.1.1.1.3 Principal compression stress limits for SEE:

Normal weight concrete	$0.30 f'_c$
------------------------	-------------

### 4.2 STRENGTH REDUCTION FACTORS

Substructure shall be designed using Load Factor Design (LFD), with Strength Reduction Factors based on Caltrans BDS 8.16.1.2.2 except for Seismic Load Combinations. See Section 7.8 of this Criteria.

### 4.3 MATERIALS

#### 4.3.1 Concrete

##### 4.3.1.1 W2 Cap Beam (Normal weight concrete)

56-day minimum strength	$f'_c = 60$ MPa	(8700 psi)
at time of stressing	$f'_c = 36$ MPa	(5220 psi)

<b>4.3.1.2 W2 Pier (Normal weight concrete)</b>		
56-day minimum strength	$f_c = 55$ MPa	(7900 psi)
<b>4.3.1.3 W2 Footing (Normal weight concrete)</b>		
90-day minimum strength	$f_c = 35$ MPa	(5000 psi)
<b>4.3.1.4 Tower Footing (Normal weight concrete)</b>		
56-day minimum strength	$f_c = 35$ MPa	(5000 psi)
<b>4.3.1.5 E2 Cross Beam (Normal weight concrete)</b>		
56-day minimum strength	$f_c = 55$ MPa	(7900 psi)
<b>4.3.1.6 E2 Pier (Normal weight concrete)</b>		
56-day minimum strength	$f_c = 35$ MPa	(5000 psi)
<b>4.3.1.7 E2 Footing</b>		
<b>Normal weight concrete</b>		
56-day minimum strength	$f_c = 35$ MPa	(5000 psi)
<b>Light weight concrete</b>		
56-day minimum strength	$f_c = 35$ MPa	(5000 psi)
<b>4.3.1.8 Retaining Walls (Normal weight concrete)</b>		
56-day minimum strength	$f_c = 35$ MPa	(5000 psi)
<b>4.3.1.9 Piles (Normal weight concrete)</b>		
<b>CIDH or CISS Concrete Piling</b>		
56-day minimum strength	$f_c = 25$ MPa	(3600 psi)
<b>CIDH Concrete Piling or CIDH Concrete Piling (Rock Socket)</b>		
56-day minimum strength	$f_c = 35$ MPa	(5000 psi)
<b>4.3.1.10 Epoxy Grout</b>		
Epoxy Grout at the Tower Base	$f_c = 104$ MPa	(15 ksi)
<b>4.3.2 Prestressing Steel</b>		
<b>4.3.2.1 Strand Prestressing Steel</b>		
15 mm AASHTO M203 (ASTM A416), uncoated seven-wire low relaxation strand Grade 270.		
<b>4.3.2.1.1 Properties</b>		
Guaranteed Ultimate Tensile strength	$f_s = 1860$ MPa	(270 ksi)
Yield strength	$f_y^* = 1674$ MPa	(243 ksi)
Modulus of elasticity	$E = 197,000$ MPa	(28,500 ksi)
Anchor set	$\Delta = 6$ mm	(1/4 in.)

4.3.2.1.2 Prestress losses

For conventional CIP Concrete on Falsework:

Standard galvanized steel ducts:

Wobble coefficient  $\kappa = 0.0007 / m$  (0.0002 /ft)  
 Friction coefficient  $\mu = 0.20$

4.3.2.2 Bar Prestressing Steel

ASTM A722 high strength rods. Standard galvanized steel ducts.

Tensile strength  $f_s = 1030 \text{ MPa}$  (150 ksi)  
 Modulus of elasticity  $E = 207,000 \text{ MPa}$  (30,000 ksi)  
 Anchor set  $\Delta = 2 \text{ mm}$  (1/12 in.)  
 Friction coefficient  $\mu = 0.25$   
 Wobble coefficient  $\kappa = 0.0007 / m$  (0.0002 /ft)

75 mm high strength PT rods. Standard galvanized steel ducts.

Tensile strength  $f_s = 1030 \text{ MPa}$  (150 ksi)  
 Modulus of elasticity  $E = 205,000 \text{ MPa}$  (29,730 ksi)  
 Anchor set  $\Delta = 2 \text{ mm}$  (1/12 in.)  
 Friction coefficient  $\mu = 0.25$   
 Wobble coefficient  $\kappa = 0.0012 / m$  (0.0004 /ft)

4.3.3 Mild Steel Reinforcement

All mild steel reinforcement shall be ASTM A706M Gr. 415. The following properties shall be used in the design:

Minimum yield strength  $f_y = 415 \text{ MPa}$  (60 ksi)  
 Maximum tensile stress  $f_u = 738 \text{ MPa}$  (107 ksi)  
 Modulus of elasticity  $E_s = 200,000 \text{ MPa}$  (29,000 ksi)

5. CORROSION PROTECTION

The structure design service life is 150 years. Corrosion allowances for exposed structural steel surfaces vary in the submerged zone. The splash zone extends from -1.47 m NGVD to 7.1 m NGVD. Exposed structural steel shall not be used in splash zone.

5.1 PILES

The pile shells shall be designed for corrosion to ensure that the minimum design thickness will be present at the end of the 150-year design life. The minimum sacrificial steel corrosion allowances to be added to the minimum structural design thickness of the shell are given below.

Piers	Depth	Top	Bottom	Corr. Allow.
Pier 1 (Tower)	Exposed zone	Top of pile	Elev. -11.000m	21 mm
Pier 1 (Tower)	Scour zone	Elev. -11.000m	Pile tip	20 mm *
E2	Immersed zone	Top of pile	Elev -12.7 m	21.4 mm
E2	Scour zone	Elev -12.7 m	Elev -16.9 m	25.5 mm
E2	Buried zone	Elev -16.9 m	Elev -31.9m	4.2 mm

\* Per CALTRANS email dated 07/23/02

The corrosion allowances shall account for any under-drive allowance. The outside diameter of the pile, including the steel corrosion allowance, shall be constant. The section of the pile embedded in the pile cap shall have the same steel corrosion allowance as the exposed upper section of the pile. No corrosion allowance shall be added to the shear connectors. Steel reinforcement inside the piles shall be uncoated.

**5.2 REINFORCEMENT PROTECTION**

For piers and footings in water, epoxy-coated reinforcement shall be used for exterior bars from the top of the splash zone to the bottom of the footing. Epoxy coating for pier and footing reinforcement shall be purple epoxy conforming to Caltrans Standard Special Provision (SSP) 52M\_PURP. This shall include the first exterior mats of top, bottom, and side-face pile cap reinforcement.

**5.3 BARRIERS**

Epoxy-coated reinforcement shall be used in all concrete barrier railings.

**5.4 CORROSION MONITORING SYSTEM AND CONTINGENCY CATHODIC PROTECTION**

The pile shell corrosion monitoring system and electrical connections for contingency cathodic protection are not covered in this document.

## 6. STRUCTURAL STEEL

### 6.1 MATERIALS

Unless modified herein, or specified in the BDS, structural steel shall comply with the AASHTO and ASTM Materials Specifications.

#### 6.1.1 Structural Steel

The following steels shall be used:

ASTM A709M Gr. 345, ASTM A709M HPS 485W, ASTM A514M Gr. 690,  
 and Shear Link Gr. 345

Shear Link Gr. 345

Yield Strength, min.	345 MPa (50 ksi)
Yield Strength, max.	380 MPa (55 ksi)

#### 6.1.2 Structural Steel Connection

High Strength Bolts ASTM A325-X, ASTM A490-X

Main Cable Strand Anchor Rod,  
 Suspender Socket Anchor Rod, East  
 Saddle Tie Rod, East Saddle Anchor  
 Rods, Tower Anchor Bolts, Tower  
 Saddle Tie Rods, and Pier E2 Bearing  
 and Shear Key Anchor Bolts ASTM A354 Gr. BD

Cable Band Bolts, West Deviation Saddle  
 Anchor Rods, West Deviation Saddle  
 Bolts, and West Jacking Saddle Bolts ASTM A354 Gr. BC

Dowels ASTM A633 Gr. E

Cap Screws ASTM A240 Type 316

#### 6.1.3 Main Cable

Parallel zinc coated carbon steel wire, diameter 5.40 mm (including coating)

Tensile Strength, min.	1,760 MPa	(254 ksi)
Yield Point, min.	1,350 MPa	(195 ksi)
Proportional Limit, min.	900 MPa	(131 ksi)
Design elastic modulus	200,000 MPa	(29,000 ksi)
Zinc coating	Class A	

#### 6.1.3.1 Suspenders

Wire Rope ASTM A603 – Class A zinc  
 coating

Wire Strength, min.	1,350 MPa	(195 ksi)
Design elastic modulus	138,000 MPa	(20,000 ksi)

#### 6.1.3.2 Wrapping Wire

ASTM A510 Gr. 1010

Tensile Strength, min.	450 MPa	(65 ksi)
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**6.1.3.3 Handrope**

Galvanized structural strands ASTM A586

**6.1.4 Castings**

Tower Saddle, East Saddle, Cable Bands,  
 Suspender Separators, Pier E2 Shear Key  
 Stub, and Pier E2 Bearing Top and  
 Bottom Housing ASTM A148M Gr. 550 – 345

West Deviation Saddle, West Jacking  
 Saddle, and Suspender Sockets/Cable  
 Strand Sockets ASTM A148M Gr. 620 – 415

Pier E2 Shear Key Housing ASTM A148M Gr. 725 – 585

**6.1.5 Platform, Ladders, and Accessories**

**6.1.5.1 Platform and Catwalk Structural Steel**

Platform and Catwalk ASTM A709M Gr. 345

High Strength Bolts ASTM A325

**6.1.5.2 Platform, Catwalk, Grating, Ladders, and Accessories (Miscellaneous Metal)**

Grating Per Manufacturer

Railing Pipe ASTM A53

Traveler Spherical Bushing Per Manufacturer

Pin (Stainless) ASTM A276

**6.2 SERVICE LOAD DESIGN**

The SFOBB-SAS bridge structural steel components shall be designed in accordance with BDS-ASD provisions.

**6.2.1 Suspension Structural System**

**6.2.1.1 Main Cables**

Allowable stress 689 MPa (100 ksi)

Allowable fatigue Stress Range 179 MPa (26 ksi)

Design void ratio at cable bands 20%

**6.2.1.2 Suspenders and Sockets**

Allowable stress 0.25 times the breaking strength

Allowable fatigue stress range 179 MPa (26 ksi)

Socket strength 110% of rope breaking strength

**6.2.1.3 Cable Bands and Bolts**

Design friction coefficient	0.11	
Allowable combined hoop and bending stress	0.6 F <sub>y</sub>	
Allowable cable band bolt stress	600 MPa	(87 ksi)
Design cable band bolt stress for friction	200 MPa	(29 ksi)

**6.2.1.4 Saddles**

Design friction coefficient	0.30	
Trough, min. primary radius	8 x Nominal cable diameter	
Vertical galvanized steel spacers	10 mm, min.	
Allowable principal stresses	0.6 F <sub>y</sub>	

"Hasegawa et al" Paper  
 "Takena et al" Paper

**6.2.1.5 Wrapping Wire**

Galvanized Wire		
Tension, min.	150 MPa	(22 ksi)
Allowable stress	0.6 F <sub>y</sub>	

**6.2.2 Orthotropic Deck**

Deck plate thickness, min.	14 mm	(5/9 in.)
Epoxy asphalt overlay (or equivalent) thickness	50 mm	(2 in.)
Deflection limitations:		
Deflection of deck plate	Span length /300	
Deflection of ribs	Span length /1000	
Relative live load deflection between adjacent ribs	2.5 mm	(0.1 in.)

"AASHTO LRFD",  
 Sect. 9.8.3.7

**6.2.3 Steel Box Girder**

Deflection limitations:		
Deflection of girder	Span length /300	

**6.2.4 Suspender Exchange and/or Loss**

The bridge shall be designed for two possible conditions involving the exchange of loss of suspenders:

1. Exchange, removal, or loss of any one suspender. The bridge shall meet the normal design criteria under reduced live load.
2. Breakage of one suspender. The bridge shall remain stable and repairable under a suspender tension equal to the breaking strength of the rope.

**6.2.5 Expansion Joints**

Bridge expansion joints shall be designed for movement demands from temperature rise and fall, creep and shrinkage, functional evaluation earthquake and safety evaluation earthquake. Temperature movements will not be combined with seismic movements. Creep and shrinkage movements shall be increased by 50%.

The modular joint seal assemblies shall be designed in accordance with the following criteria:

Service

$$0 \leq r \leq 75mm$$

where  $r$  is the unsupported length of a strip seal supported by the adjacent beams

Functional Evaluation Earthquake

$$0 \leq r \leq 130mm$$

Safety Evaluation Earthquake

Closing:  $0 \leq r$

Opening: Support bar shall not be unseated

Bike path expansion joints shall be designed for movement demands from temperature rise and fall and creep and shrinkage.

## 7. SEISMIC DESIGN

Seismic design shall be performed in accordance with BDS, modified by or augmented with pertinent provisions of ATC-32 and project specific criteria as detailed in this document.

### 7.1 SEISMIC LOADING

#### 7.1.1 *Design Seismic Loading*

The bridge design shall be based on nonlinear dynamic analysis for three sets of spectrum-compatible tri-component multiple-support time histories for both an event on the Hayward fault and an event on the San Andreas fault. The maximum response from the six SEE sets of ground motions shall be used for design. The bridge shall also be checked for an FEE using nonlinear dynamic analysis.

"Seismic Ground Motion"  
Report

#### 7.1.2 *Seismic Loading During Construction*

The bridge shall be designed to resist an equivalent static load of 0.1g for configurations occurring under the assumed construction sequence.

### 7.2 PERFORMANCE CRITERIA

The bridge shall be designed to provide a high level of seismic performance. It shall be designed to resist two levels of earthquake:

- A functional evaluation earthquake
- A safety evaluation earthquake

#### 7.2.1 *Functional Evaluation Earthquake*

After a functional evaluation earthquake the bridge will provide:

- Full service almost immediately, with only minimal damage to the structure

"SDCB"

Minimal damage implies essentially elastic performance, and is characterized by:

- Minor inelastic response
- Narrow cracking in concrete
- No apparent permanent deformations
- Damage to expansion joints that can be temporarily bridged with steel plates

**7.2.2 Safety Evaluation Earthquake**

After a safety evaluation earthquake the bridge will provide:

- Full service almost immediately, with no more than repairable damage to the structure

Repairable damage is damage that can be repaired with a minimum risk, such as:

- Minimal damage to superstructure and tower shaft
- Limited damage to piers (including yielding of reinforcement and spalling of concrete cover) and tower shear links
- Minimal damage to Piles and Pile caps
- Small permanent deformations, not interfering with serviceability of the bridge
- Damage to expansion joints that can be temporarily bridged with steel plates

**7.2.3 Limited Ductility Structure**

The bridge shall be designed as a limited ductility structure (as defined in ATC-32) with stable response to ground motions equivalent to the safety evaluation earthquake. The stability of the structure shall be demonstrated by means of pushover analysis or an equivalent method of structural evaluation. This means:

- The bridge shall have a clearly defined inelastic mechanism for response to lateral loads
- Inelastic behavior shall be restricted to piers, tower shear links, and hinge beam fuses
- The detailing and proportioning requirements for full-ductility structures as defined in ATC-32 shall be met

**7.2.4 Collapse Avoidance**

For seismic loading during construction, the performance objective shall be to avoid collapse.

"SDCB"

### 7.3 SEISMIC ANALYSIS

#### 7.3.1 *Design Seismic Loading*

The self-anchored suspension bridge shall be designed using nonlinear time history analysis (which accounts for large displacement and inelastic material behavior). Preliminary design for the bridge may utilize elastic time history analysis with a Z-factor of 2.0.

#### 7.3.2 *Displacement-Based Design*

Pushover analysis shall be used to calculate the displacement capacity of the structural components. The displacement capacity, corresponding to the limiting strains in materials, must exceed or be equal to the calculated demand.

The displacement capacity of the structural component shall be evaluated using the limiting material strains discussed in Section 7.11 of this document.

#### 7.3.3 *Soil-Structure Interaction Effects*

Soil-structure interaction effects shall be considered in all analyses. Foundation dynamic characteristics shall be incorporated into the analysis with discrete elements representing piles and footings with appropriate representation of the effects of soil structure interaction (SSI). Depth varying free-field motions shall be applied to the analysis model along the buried length of the appropriately discretized piles.

"SDCB"

### 7.4 PERMANENT DISPLACEMENTS

To ensure the ability of the bridge to carry traffic across expansion joints after a safety evaluation earthquake, the estimated permanent displacement of the bridge, as established by nonlinear dynamic analysis, on average shall not exceed 300 mm (11.8 inches) at the deck level.

To ensure the structural integrity and serviceability of the bridge after the SEE, permanent vertical settlements at the pile caps shall not exceed 50 mm.

"ATC-32", 3.21.10.3

### 7.5 CAPACITY DESIGN

Significant inelastic deformations are expected to occur in the following elements of the SFOBB SAS: the west pier columns, the east pier, and the tower shear links. Structural elements constituting the ductile lateral load resisting part of the structure shall be designed to resist the over strength capacity of the elements undergoing inelastic deformations. Components of the SFOBB SAS which this section applies to are: W2 closure joint, W2 cap beam, E2 shear key, bearings, E2 cap beam, pile caps, and tower shear link connections.

"ATC-32", 3.21.14,  
10.19.3, 10.20, 10.24.8

### 7.6 HINGE BEAMS

Hinge beam frames shall be proportioned for the forces and displacements calculated by the appropriate non-linear dynamic time history analysis of the connected frames.

### 7.7 RESTRAINING FEATURES

Restraining devices connecting structural components or frames shall be proportioned for the forces and displacements calculated by non-linear dynamic time history analysis of the connected structural components or frames.

Dampers or other energy dissipation devices may be considered for the reduction of relative displacement between structural components and frames.

"ATC-32", 3.21.12,  
10.29.8

### 7.8 STRENGTH REDUCTION FACTORS FOR COLUMNS

Strength reduction factors shall be per ATC-32.

"ATC-32", 8.16.1.2.2

### 7.9 DESIGN

#### 7.9.1 Substructure

##### 7.9.1.1 Design flexural strength

The design of sections for flexure shall be based on expected material strengths:

$$\begin{aligned}f_{ce} &= 1.3 f_c \\f_{ye} &= 1.1 f_y \\F_{ye} &= 1.1 F_y\end{aligned}$$

Maximum concrete strains at the design flexural strength shall not exceed 0.004. If moment curvature analysis is used to determine the design flexural strength, the steel strains shall be limited to 0.015.

"ATC-32", 8.16.2.4.1

##### 7.9.1.2 Maximum plastic moment

The maximum plastic moment shall be calculated using one of the following two methods:

Maximum plastic moments shall be determined from moment-curvature analysis that considers the effects of concrete confinement and strain hardening of the reinforcement. The maximum plastic moment shall be taken at the design deformation of the element.

1. The maximum plastic moment of sections shall be based on maximum feasible material strength:

$$\begin{aligned}f_{co} &= 1.7 f_c \\f_{yo} &= 1.25 f_y \\F_{yo} &= 1.25 F_y\end{aligned}$$

Maximum plastic moments shall be determined from moment-curvature analysis that considers the effects of concrete confinement and strain hardening of the reinforcement. The maximum plastic moment shall be the calculated moment at the design deformation of the element.

"ATC-32", 8.16.4.4,  
10.63

2. The maximum plastic moment of sections shall be based on expected material strength:

$$\begin{aligned}f_{ce} &= 1.3 f_c \\f_{ye} &= 1.1 f_y \\F_{ye} &= 1.1 F_y\end{aligned}$$

Maximum plastic moments shall be determined from moment-curvature analysis that considers the effects of concrete confinement and strain hardening of the reinforcement. The maximum plastic moment shall be at least 1.15 times the calculated moment at the design deformation of the element.

### 7.9.2 Main Tower

#### 7.9.2.1 Tower Shear Links

The tower shear links shall be designed in accordance with "AISC – Seismic Provisions for Structural Steel Buildings."

##### 7.9.2.1.1 Design Shear Strength

The design of sections for shear shall be based on expected material strengths:

$$F_{ye} = R_y F_y \quad \text{where } R_y = 1.1$$

"AISC – Seismic Provisions", Section 6.2

##### 7.9.2.1.2 Maximum Plastic Shear

The over-strength of sections shall be based on maximum feasible material strengths:

$$F_{y0} = \beta F_{ye} \quad \text{where } \beta = 1.25$$

"AISC – Seismic Provisions", Section 15.6

##### 7.9.2.1.3 Shear Link Connection

In accordance with Section 7.5, the shear link connection shall be designed for the over-strength of the shear link. AISC-LRFD requirements shall apply as follows:

$$\phi R_n \geq R_y \beta V$$

$\phi$  = Reduction factor for a given failure mode  
 $R_n$  = Nominal resistance capacity for a given failure mode  
 $R_y$  = Expected material strength factor  
 $\beta$  = Over-strength factor  
 $V$  = Force associated with the shear yield of the shear link

### 7.9.2.2 Tower Shafts

The tower shafts shall be designed in accordance with ATC-32 augmented by the following requirements:

$$\eta / \eta_{req} > 2.0$$

$$b/t \leq 21$$

$$P_{max} / Area \leq 0.6 F_y$$

where:

$\eta$  = is the relative stiffness of the longitudinal stiffener to the tower skin wall

$b/t$  = is the width to thickness ratio of the tower skin wall

$P_{max} / Area$  = is the maximal axial stress

The tower shafts shall be designed to remain almost elastic during the SEE.

"ATC-32", 10.63.3

### 7.9.2.3 Tower Connections

#### 7.9.2.3.1 Tower Splices

The tower splices (tower shaft splices and tower head splices) shall be designed for the expected yield strength capacity of the connected component in accordance with AASHTO-LRFD.

#### 7.9.2.3.2 Tower Anchorage

The tower anchorage shall be designed based on a global push-over of the tower. The capacity of the tower anchorage shall be larger than the demands associated with the plastic hinging of the tower shaft. The capacity shall be evaluated in accordance with AASHTO-LRFD.

**7.9.3 Superstructure**

**7.9.3.1 Box Girder and Cross Beams**

The box girder and cross beams shall be designed in accordance with BDS augmented by the following requirements:

$$\eta/\eta_{req} > 1.0$$

$$b/t \leq 27$$

$$d_l/d_{lreq} > 1.0$$

where:

$\eta$  = is the relative stiffness of the longitudinal

$b/t$  = is the width to thickness ratio

$d_l$  = is the ratio of the cross sectional area of the longitudinal stiffener

The box girder and cross beams shall be designed to remain almost elastic during SEE.

**7.9.3.2 Connections**

All connections shall be designed for the yield strength capacity of the connected component in accordance with BDS-LFD. For connections containing welded portions, the bolted connections shall be designed as slip critical (LFD method). Otherwise, the bolted connection shall be designed as bearing type connection.

**7.9.4 Suspension Structural System**

**7.9.4.1 Cable and Suspenders**

Allowable tensile stresses  $0.5 F_u$

**7.9.4.2 Saddles**

Allowable principal stresses  $1.0 F_y$

**7.10 DEFORMATION CAPACITY**

The deformation capacity of structural members shall be calculated using the allowable material strains from Section 7.11, and, where applicable, using plastic hinge lengths calculated according to ATC-32 8.18.2.4.2.

For the piers, the material strains shall be assumed to be the average extreme fiber strains over the plastic hinge length. For the piles, the strains shall be assumed to be the maximum extreme fiber strains.

Plastic hinge lengths for hollow reinforced concrete sections and pile sections shall be verified by laboratory testing and/or detailed non-linear analysis.

"ATC-32", eq. R10-12

BDS eq. 10-131

"ATC-32", eq. R10-14

**7.11 ALLOWABLE STRAINS**

**7.11.1 Normal weight concrete**

Allowable strains in normal weight concrete shall be:

Piers (Average extreme fiber strains in plastic hinge):

Functional evaluation earthquake  $\epsilon_c^{FEE} = 0.004$

Safety evaluation earthquake  $\epsilon_c^{SEE} = 2/3 \epsilon_{cu}$

where  $\epsilon_{cu}$  is the ultimate concrete strain according to the Mander model

(Mander et.al. J. Struct. Engineering, ASCE, 1988 114(8), 0 1804 – 1849)

Piles (Maximum extreme fiber strains):

Functional evaluation earthquake  $\epsilon_c^{FEE} = 0.004$

Safety evaluation earthquake  $\epsilon_c^{SEE} = 0.01$

**7.11.2 Mild Steel Reinforcement**

Allowable strains in mild steel reinforcement shall be:

Piers (average extreme fiber strains in plastic hinge):

Functional evaluation earthquake  $\epsilon_s^{FEE} = 0.015$

Safety evaluation earthquake  $\epsilon_s^{SEE} = 2/3 \epsilon_{su}$

where  $\epsilon_{su}$  is the steel strain at ultimate stress.

"ATC-32", C3.21.11.1

Piles (maximum extreme fiber strains in potential plastic hinge):

Functional evaluation earthquake  $\epsilon_s^{FEE} = 0.015$

Safety evaluation earthquake  $\epsilon_s^{SEE} = 0.02$

For ASTM A706M Gr. 415 reinforcement,  $\epsilon_{su}$  may be taken as:

Confinement bars No. 10 – 25 (No. 3 – 8)  $\epsilon_{su} = 0.12$

Main bars No. 29 – 57 (No. 9 – 18)  $\epsilon_{su} = 0.09$

Hardening strains may be taken as:

Bars No. 10 – 25 (No. 3 – 8)  $\epsilon_{sh} = 0.0150$

Bars No. 29 – 36 (No. 9 – 11)  $\epsilon_{sh} = 0.0100$

Bar No. 43 (No. 14)  $\epsilon_{sh} = 0.0075$

Bar No. 57 (No. 18)  $\epsilon_{sh} = 0.0050$

**7.11.3 Structural Steel Pile Shells (Casings)**

Functional evaluation earthquake  $\epsilon_s^{FEE} = 0.015$

Safety evaluation earthquake  $\epsilon_s^{SEE} = 0.02$

Hardening strain may be taken as  $\epsilon_{sh} = 0.018$

**7.11.4 Tower Shafts**

In case of over-load, the allowable tower shaft strain shall be:

$$\epsilon_s^{\text{overload}} = 4 \cdot \epsilon_y \quad \text{where } \epsilon_y \text{ is the yield strain of the steel}$$

"Kawashima et al"  
Report

**7.11.5 Tower Shear Links**

Maximum rotation,  $\gamma = 0.08 \text{ rad}$

"AISC – Seismic Provisions", Section 15.2g

**7.11.6 Box Girder and Cross Beams**

In case of over-load, the allowable strain for the box girder and cross beams is:

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$$\epsilon_s \text{ overload} = 2 * \epsilon_y \quad \text{where } \epsilon_y \text{ is the yield strain of the steel}$$

**7.12 DESIGN FOR JOINT SHEAR**

Cap beam/pier joints, pier/footing joints, and other joints between reinforced concrete flexural members shall be proportioned and reinforced for joint shear in accordance with Section 8.34 of ATC-32.

"ATC-32", 8.34

**7.13 COLUMN SHEAR**

All columns shall be designed to resist the maximum plastic shear obtained using the maximum plastic moment capacity of the column. Shear capacity shall be determined according to Section 3.6 of Version 1.1 of the Caltrans Design Criteria.

**8. FOUNDATION DESIGN**

Geotechnical parameters used in the design will be those provided by Fugro and Earth Mechanics for this project.

"Axial Pile and Drivability"  
Report

"Lateral Pile Design"  
Report



**San Francisco-Oakland Bay Bridge  
East Span Seismic Safety Project  
Contract 59A0040**

***DESIGN CRITERIA***  
**Skyway Structures**  
**March 1, 2001**

Prepared by T.Y.Lin International/Moffatt & Nichol Engineers, a Joint Venture

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1. GENERAL

The bridge shall be designed in accordance with "Caltrans Bridge Design Specifications Manual (1995) (BDS)," modified or augmented as detailed in this design document.

In addition to bridge and site specific criteria, pertinent sections of the following standards or codes have been employed for such modifications or augmentations.

- "AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges", 1999 Edition
- "Proposed LRFD Guide Specifications for Design of Segmental Concrete Bridges", S. I. Units, March 1997
- "AASHTO Standard Specifications for Highway Bridges", 16<sup>th</sup> Edition, 1992
- "AASHTO LRFD Bridge Design Specifications" 2<sup>nd</sup> Edition, 1998.
- "Sacramento Light Rail Transit Design Criteria", May 1993
- "San Francisco-Oakland Bay Bridge East Span Seismic Safety Project Light Rail Transit Design Criteria", 1999
- "Improved Seismic Design Criteria for California Bridges: Provisional Recommendations" ATC-32 Report, June 30, 1996
- "Proposed Design Specifications for Steel Box Girder Bridges", FHWA-TS-80-205, FHWA, Washington, DC, 1980
- "Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges" Volume I: Final Report. February, 1991
- "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Working Stress Design", API RP2A-WSD 20<sup>th</sup> Ed. 1993
- Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design – API RP2A-LRFD 1<sup>st</sup> Ed. 1993
- AISC Manual of Steel Construction, 9<sup>th</sup> Edition
- ANSI/ASCE 7-95 Standard
- Stability Design Criteria – 5<sup>th</sup> Edition, SSRC
- "SFOBB East Spans; Seismic Design Criteria Basis,"(SDCB) June 27, 2000
- Technical Specifications for Skyway Structures

"AASHTO Seg."

"LRFD Seg."

"AASHTO"

"AASHTO LRFD"

"Sac. LRT"

"SFOBB LRT"

"ATC-32"

Brian Maroney

Contract Documents

**2. DESIGN LOADS**

This section covers all design loads except for seismic demands discussed in Section 7.

**2.1 STRUCTURAL DEAD LOADS – DL**

Unless specified herein or in BDS, all loads shall be as specified in the Specifications cited in Article 1., General.

**2.1.1 Concrete**

The in-service air dry unit mass of normal weight concrete, including reinforcement shall be  $2480 \text{ kg/m}^3$  ( $155 \text{ lb/ft}^3$ ).

The in-service air dry unit mass of sand lightweight concrete including reinforcement shall be  $1920 \text{ kg/m}^3$  ( $120 \text{ lb/ft}^3$ ).

**2.1.2 Steel**

The unit mass of structural steel, including fabricated plate steel, rolled shapes, and wire, shall be  $7850 \text{ kg/m}^3$  ( $490 \text{ lb/ft}^3$ ).

**2.2 OTHER PERMANENT LOADS- SDL**

Other permanent loads are assumed to be applied at the time of construction except for utilities and dead load of Light Rail Transit (LRT) appurtenances.

**2.2.1 Vehicular and Pedestrian Barriers**

Concrete roadway barrier

Modified 732 barrier 8.5 kN/m (585 lb/ft)

Steel barrier 2.73 kN/m (188 lb/ft)

Bike path separation rail and pedestrian handrail 0.60 kN/m (40 lb/ft)

**2.2.2 Wearing Surface**

Skyway Steel Transition Span – 50 mm 1.18 kN/m<sup>2</sup> (25 lb/ft<sup>2</sup>)

Bike path– 13 mm 0.31 kN/m<sup>2</sup> (6 lb/ft<sup>2</sup>)

Skyway Concrete Box Girder – 20 mm 0.47 kN/m<sup>2</sup> (10 lb/ft<sup>2</sup>)

**2.2.3 Utilities:**

East bound structure 12 kN/m (820 lb/ft)

West bound structure 10 kN/m (685 lb/ft)

**2.3 LIVE LOADS – LL+I**

Live loads shall be considered for two roadway configurations:

- Highway traffic (6 lanes) on the Westbound Structure; highway traffic (6 lanes) plus a bicycle/pedestrian facility on the south side of the Eastbound Structure.
- An LRT system on the inside lane plus highway traffic (4 lanes) on the Westbound Structure; an LRT system on the inside lane plus highway traffic (4 lanes) on the Eastbound Structure with a bicycle/pedestrian facility on the south side.

**2.3.1 Standard Truck and Lane Loads**

Vehicular live load shall be AASHTO HS20-44.

All design live loads shall be positioned both transversely and longitudinally

so as to produce the maximum influence on the structure.

In the design of the orthotropic deck, one axle load of 107 kN (24 kips) or two axle loads of 71 kN (16 kips) each spaced 1.2 m (4-ft) apart, whichever produces the greater stress, shall be used.

**2.3.2 Load reduction factors for multiple lane loading**

Load reduction factors for multiple lane loading shall be applied to highway lanes only. Full LRT Train Loading shall be combined with highway traffic loading to produce the most critical stress condition in the member.

**2.3.3 Permit Vehicle Loads**

Permit loads (P13) shall be considered for design of the bridge superstructure.

BDS 3.7.7, Figure 3.7.7B and 3.11.4.3

Permit loads (P13) will not be applicable to the orthotropic deck design.

BDS Figure 3.7.7A

**2.3.4 Pedestrian Loads**

Pedestrian loading shall be in accordance with BDS.

BDS 3.14.1

**2.3.5 Bike-Path Vehicle Loading**

Bike path deck shall be designed for an H-15 wheel load including impact.

**2.3.6 Live Load Contribution Under Seismic Conditions**

A reduced live load shall be considered for combination with seismic demands (Group VIIA loading). The reduced loading factor  $\beta_{LE}$  shall represent estimated peak hour traffic predictions for the year 2025.

<p><b>2.4 THERMAL EFFECTS – T</b></p> <p><b>2.4.1 Uniform Temperature – <math>T_c</math> and <math>T_s</math></b>                  Design temperature range shall correspond to BDS requirements for coastal areas:</p> <table border="0" style="margin-left: 40px;"> <tr> <td style="padding-right: 20px;">Mean Temperature Rise or Fall</td> <td style="padding-right: 40px;">Concrete</td> <td style="padding-right: 40px;">Steel</td> <td style="padding-right: 40px;">27° C (81° F)</td> <td style="padding-right: 40px;">17° C (30° F)</td> <td style="padding-right: 40px;">22° C (40° F)</td> </tr> </table> <p><b>2.4.2 Coefficient of thermal expansion</b></p> <table border="0" style="margin-left: 40px;"> <tr> <td style="padding-right: 20px;">Normal-weight Concrete</td> <td style="padding-right: 20px;">Sand Light-weight Concrete</td> <td style="padding-right: 20px;">Steel</td> <td style="padding-right: 20px;">10.8x10<sup>-6</sup> /°C</td> <td style="padding-right: 20px;">9.0x10<sup>-6</sup> /°C</td> <td style="padding-right: 20px;">11.7x10<sup>-6</sup> /°C</td> <td style="padding-right: 20px;">(6.0x10<sup>-6</sup> /°F)</td> <td style="padding-right: 20px;">(5.0x10<sup>-6</sup> /°F)</td> <td style="padding-right: 20px;">(6.5x10<sup>-6</sup> /°F)</td> </tr> </table> <p><b>2.4.3 Concrete Temperature gradient - <math>DT_c</math></b>                  Positive and negative temperature gradients, diagrammed on Fig. 6.4 of the AASHTO Guide Specifications for Segmental Concrete Bridges, shall correspond to the values listed below.</p> <table border="0" style="margin-left: 40px; width: 100%;"> <thead> <tr> <th style="text-align: left; width: 50%;"><u>Positive Gradient</u></th> <th style="text-align: left; width: 50%;"><u>Negative Gradient</u></th> </tr> </thead> <tbody> <tr> <td><math>T_1 = 30.0^\circ \text{ C (54}^\circ \text{ F)}</math></td> <td><math>T_1 = -15.0^\circ \text{ C (-27}^\circ \text{ F)}</math></td> </tr> <tr> <td><math>T_2 = 7.8^\circ \text{ C (14}^\circ \text{ F)}</math></td> <td><math>T_2 = -3.9^\circ \text{ C (-7}^\circ \text{ F)}</math></td> </tr> <tr> <td><math>T_3 = 0^\circ \text{ C (0}^\circ \text{ F)}</math></td> <td><math>T_3 = 0^\circ \text{ C (0}^\circ \text{ F)}</math></td> </tr> <tr> <td><math>A = 300 \text{ mm (12 in)}</math></td> <td><math>A = 300 \text{ mm (12 in)}</math></td> </tr> </tbody> </table> <p><b>2.4.4 Steel Temperature gradient - <math>DT_s</math></b>                  For steel structures, the gradient shall be as shown in Figure 3.12.3.2 of the AASHTO LRFD Specifications for Highway Bridge Design.</p> <p>A temperature gradient of 11° C (20° F) between the top and bottom of the box will be considered for global stresses in the design of the orthotropic deck.</p> <p>Negative temperature gradients need not be considered.</p>	Mean Temperature Rise or Fall	Concrete	Steel	27° C (81° F)	17° C (30° F)	22° C (40° F)	Normal-weight Concrete	Sand Light-weight Concrete	Steel	10.8x10 <sup>-6</sup> /°C	9.0x10 <sup>-6</sup> /°C	11.7x10 <sup>-6</sup> /°C	(6.0x10 <sup>-6</sup> /°F)	(5.0x10 <sup>-6</sup> /°F)	(6.5x10 <sup>-6</sup> /°F)	<u>Positive Gradient</u>	<u>Negative Gradient</u>	$T_1 = 30.0^\circ \text{ C (54}^\circ \text{ F)}$	$T_1 = -15.0^\circ \text{ C (-27}^\circ \text{ F)}$	$T_2 = 7.8^\circ \text{ C (14}^\circ \text{ F)}$	$T_2 = -3.9^\circ \text{ C (-7}^\circ \text{ F)}$	$T_3 = 0^\circ \text{ C (0}^\circ \text{ F)}$	$T_3 = 0^\circ \text{ C (0}^\circ \text{ F)}$	$A = 300 \text{ mm (12 in)}$	$A = 300 \text{ mm (12 in)}$	<p>BDS 3.16</p> <p>AASHTO Seg. 6.4.3</p> <p>AASHTO Seg. 6.4.4 Figure 6.4</p> <p>AASHTO LRFD 3.12.3</p> <p>BDS 3.18</p> <p>BDS 3.15</p> <p>BDS 3.15 ANSI/ASCE 7-95</p>
Mean Temperature Rise or Fall	Concrete	Steel	27° C (81° F)	17° C (30° F)	22° C (40° F)																					
Normal-weight Concrete	Sand Light-weight Concrete	Steel	10.8x10 <sup>-6</sup> /°C	9.0x10 <sup>-6</sup> /°C	11.7x10 <sup>-6</sup> /°C	(6.0x10 <sup>-6</sup> /°F)	(5.0x10 <sup>-6</sup> /°F)	(6.5x10 <sup>-6</sup> /°F)																		
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$A = 300 \text{ mm (12 in)}$	$A = 300 \text{ mm (12 in)}$																									
<p><b>2.5 STREAM FLOW – SF</b>                  Forces due to stream flow shall be applied to each substructure in accordance with BDS, using the 100-year return period pool elevation and velocities from the project Hydraulics Report.</p>	<p>BDS 3.18</p>																									
<p><b>2.6 WIND – W</b>                  The Skyway shall be designed for wind loads as per BDS and ANSI/ASCE 7-95.</p> <p>These specifications provide loads for a wind event having a probability consistent with a mean return period of 50 years. For this project, the BDS and ANSI/ASCE loads shall be adjusted for a wind having a probability consistent with a mean return period of 100 years.</p> <p>In this event, the nominal stresses and/or loads and resistance factors of BDS</p>	<p>BDS 3.15</p> <p>BDS 3.15 ANSI/ASCE 7-95</p>																									

shall be met by all bridge components.

**2.7 SHIP COLLISION – SC**

“AASHTO Guide Specifications and Commentary for Vessel Collision Design,” Volume I, Final Report, February 1991 will be used to determine the ship impact force.

The bridge is classified as “critical bridge” and the acceptable annual frequency of collapse shall be equal to or less than 0.01 in 100 years. The design vessel shall be selected based on Method II.

AASHTO Vessel Collision  
Design of Highway  
Bridges

**2.8 COMBINATION OF LOADS**

**2.8.1 *Service and Ultimate Limit States***

Loading combinations shall be in accordance with Caltrans BDS Tables 3.22.1A and 3.22.1B, AASHTO “Guide Specifications for Design and Construction of Segmental Concrete Bridges” and “Sacramento Regional Transit District Light Rail Design Criteria”.

AASHTO Seg 7.0

Tables 2.8.1A and 2.8.1B show the applicable load combinations for Service Load Design and Load Factor Design.

Dead Load - D – shall include structural dead loads (DL), other permanent loads (SDL) and erection loads (EL) where applicable.

AASHTO Seg 7.2

Permanent effects of creep and shrinkage shall be added to all Service Load Design combinations with a load factor of 1.0.

AASHTO Seg 7.1

When checking tensile stresses for Service Load Design the variable load effects shall be divided by the allowable stress increases.

AASHTO Seg 7.0

One half of the temperature gradient shall be used for load combinations that include full live load plus impact.

Group Loading Combinations for Service Load Design and Load Factor Design are given by:

$$\text{Group (N)} = \gamma \{ [\beta_d D + \beta_L(L+I) + \beta_c CF + \beta_E E + \beta_B B + \beta_{SF} SF + \beta_W W + \beta_{PS} PS + \beta_{WL} WL + \beta_{LF} LF + \beta_R(R+S+T+DT) + \beta_{EQ} EQ] + \alpha_{LRT} [\beta_{L,LRT} (L_{LRT} + I_{LRT}) + \beta_{CF,LRT} CF_{LRT} + \beta_{LF,LRT} LF_{LRT} + \beta_{WL,LRT} WL_{LRT}] \}$$

where

N	Group No.
$\gamma$	Load Factor, see Table 2.8.1A and 2.8.1B
$\beta$	Coefficient, see Table 2.8.1A and 2.8.1B
D	Dead Load
L	Live Load
I	Live Load Impact
E	Earth Pressure
B	Buoyancy
W	Wind Load on Structure
WL	Wind on Live Load.
LF	Longitudinal Force from Live Load
CF	Centrifugal Force
R	Rib Shortening
S	Shrinkage
T	Temperature
DT	Thermal Gradient
EQ	Earthquake
SF	Stream Flow Pressure
PS	Prestress
$\alpha_{LRT}$	1 or 0, corresponding to presence or absence of LRT Train
$L_{LRT}$	LRT Live Load
$I_{LRT}$	Impact from LRT Live Load
$CF_{LRT}$	Centrifugal Force from LRT Train
$HF_{LRT}$	Horizontal Force, LRT Train
$LF_{LRT}$	Longitudinal Force, LRT Train
$WL_{LRT}$	Wind on LRT Train

**2.8.2 Additional Thermal Loading Combination – Concrete**

In addition to Service Load Design Combinations defined in Table 2.8.1A, the following load combination shall apply:

$$(DL + SDL + EL) + \beta_E E + B + SF + R + S + T + DT$$

where

DL	Dead Load,
SDL	Superimposed Dead Load
EL	Erection Load in the final state

100% Allowable Stress applies to this load combination.

**2.8.3 Construction Load Combinations – Concrete**

Allowable tensile stresses for construction load combinations shall be checked in accordance with AASHTO “Guide Specifications for Design and Construction of Segmental Concrete Bridges”, 1999.

AASHTO Seg  
7.2.2

AASHTO Seg  
7.4  
Table 7-2

**2.8.4 Construction Load Combinations, Load Factor Design Check – Concrete**

The strength provided shall not be less than required by the following combinations:

For maximum forces and moments:

$$1.1 ( DL + DIFF ) + 1.3 CE + 2 A$$

For minimum forces and moments:

$$DL + CE + 2 A$$

where

DL	Dead Load
DIFF	2% differential dead load applied to one cantilever
CE	load from specialized construction equipment
A	sudden impact from an otherwise static load

**2.8.5 Construction Considerations – Steel**

Where steel units are assumed to be erected by balanced cantilever construction, the provisions of Article 2.8.4 shall apply.

Where steel units are assumed to be erected by conventional crane picks or float-ins, their adequacy may be investigated using:

- Service Load Design at 125% of the basic allowable stresses for the final condition.
- Load Factor Design

AASHTO Seg  
7.4.1  
7.4.3

### 2.8.6 Ship Impact Load Combination

The vessel collision impact forces are combined with other loads. The group loading combination has the same format as that used in the current BDS for seismic design (Group VII) with all gamma and beta factors equal to 1.0 and the ship impact force replacing the earthquake load.

The ship impact load combination is given by the following expression:  
Group Load =  $\gamma (1.0 D + 1.0 P + 1.0 B + 1.0 SF + 1.0 E)$

where

$\gamma$	1.0 for all design methods
D	Dead Load
P	Vessel Collision Impact Force
B	Buoyancy
SF	Stream Flow Pressure
E	Earth Pressure

### 2.8.7 Additional Group Loading Combinations

Loading combinations shall be in accordance with Tables 2.8.1A and 2.8.1B. In addition, the following two load combinations shall be considered for Load Factor Design:

$$\text{Group VIIA} = \gamma (1.0 D + \beta_{LE} (L + I)_H + 1.0 E + 1.0 B + 1.0 SF + 1.0 PS + 1.0 EQ)$$

$$\text{Group XIA} = \gamma (1.0 D + 1.0 E + 1.0 B + 1.0 SF + 1.0 PS + 1.0 P)$$

where

$\gamma$	1.0 for all design methods
$\beta_{LE}$	0.17 (see Section 2.3.6)
D	Dead Load
P	Vessel Collision Impact Force
B	Buoyancy
SF	Stream Flow Pressure
E	Earth Pressure
PS	Prestress
EQ	Earthquake
$(L + I)_H$	Horizontal Live Load (Highway/Bicycle/Pedestrian/LRT)

Table 2.8.1A Service Load Design

Group	Y factor	D	L+I	CF	E	B	SF	W	WL	LF	PS	R+S	T	DT	$\alpha_{L,RT}$ $L_{L,RT}+L_{RT}$	$\alpha_{L,RT}$ $CF_{L,RT}$	$\alpha_{L,RT}$ $NF_{L,RT}$	$\alpha_{L,RT}$ $LF_{L,RT}$	$\alpha_{L,RT}$ $(WL_{L,RT})$	% Allow.
I	1.0	1	1	1	1	1	1	0	0	0	1	1	0	0	1	1	0	0	0	100
$I_{L,RT}$	1.0	1	0	0	1	1	1	0	0	0	1	1	0	0	1	1	0	0	0	100
II	1.0	1	0	0	1	1	1	1	0	0	1	1	0	0	0	0	0	0	0	125
III	1.0	1	1	1	1	1	1	0.3	1	1	1	1	0	0	1	1	1	1	1	125
IV	1.0	1	1	1	1	1	1	0	0	0	1	1	1	0.5	1	1	1	0	0	125
V	1.0	1	0	0	1	1	1	1	0	0	1	1	1	1	0	0	0	0	0	140
$V_I$	1.0	1	1	1	1	1	1	0.3	1	1	1	1	1	0.5	1	1	1	1	1	140
$V_{II}^*$	1.0	1	0	0	1	1	1	0	0	0	1	1	0	1	0	0	0	0	0	100

This portion of the Table only applies when LRT is considered.

AAASHTO Seg 7.2.2

where

- D Dead Load
- L Highway + Bicycle/Pedestrian Live Load
- $I$  Highway + Bicycle/Pedestrian Live Load Impact
- E Earth Pressure
- W Wind Load on Structure
- WL Wind on Live Load
- LF Longitudinal Force from Live Load
- CF Centrifugal Force
- R Rib Shortening
- S Shrinkage
- T Temperature
- DT Thermal Gradient
- SF Stream Flow Pressure
- PS Prestress

- $\alpha_{L,RT}$  1 or 0 corresponding to presence or absence of LRT Train
- $L_{L,RT}$  LRT Live Load
- $I_{L,RT}$  Impact from LRT Live Load
- $CF_{L,RT}$  Centrifugal Force from LRT Train
- $LF_{L,RT}$  Longitudinal Force, LRT Train
- $NF_{L,RT}$  Nosing Force, LRT Train
- $WL_{L,RT}$  Wind on LRT Train

\* ASBI 8.2.2: Factors for erection loads (EL) at the final state of completion are not included in this Table. SFRAME analysis automatically accounts for these loads.

Table 2.8.1B Load Factor Design

Group	$\gamma$ factor	D	(L+I)H	(L+I)P	CF	E	B	SF	W	WL	LF	PS	R+S	T	DT	EQ	$\alpha_{LRT}$ $I_{LRT} + I_{LRT}$	$\alpha_{LRT}$ CF <sub>LRT</sub>	$\alpha_{LRT}$ NF <sub>LRT</sub>	$\alpha_{LRT}$ LF <sub>LRT</sub>	$\alpha_{LRT}$ WL <sub>LRT</sub>	RT <sub>1</sub> or BR	LR <sub>2</sub> or BR	$\alpha_{LRT}$ DR <sub>LRT</sub> + I
I	1.30	$\beta_D$	1.67	0	1	$\beta_E$	1	1	0	0	0	0.77	0	0	0	0	1	1	0	0	0	0	0	0
I <sub>spD</sub>	1.30	$\beta_D$	1	1.25	1	$\beta_E$	1	1	0	0	0	0.77	0	0	0	0	1	1	1	0	0	0	0	0
II	1.30	$\beta_D$	0	0	0	$\beta_E$	1	1	1	0	0	0.77	0	0	0	0	0	0	0	0	0	0	0	0
III	1.30	$\beta_D$	1	0	1	$\beta_E$	1	1	$\beta_{wl}$	1	1	0.77	0	0	0	0	1	1	1	1	1	1	0	0
IV	1.30	$\beta_D$	1	0	1	$\beta_E$	1	1	0	0	0	0.77	1	1	0.5	0	1	1	0	0	0	1	1	0
V	1.25	$\beta_D$	0	0	0	$\beta_E$	1	1	1	0	0	0.80	1	1	1	0	1	0	0	0	0	1	1	0
VI	1.25	$\beta_D$	1	0	1	$\beta_E$	1	1	$\beta_{wl}$	1	1	0.80	1	1	0.5	0	1	1	1	1	1	1	1	0
VII	1.00	1.0	0	0	0	$\beta_E$	1	1	0	0	0	1	0	0	1	1	0	0	0	0	0	0	0	0
XI	1.30	$\beta_D$	1	0	1	$\beta_E$	1	1	0	0	0	0.77	0	0	0	0	0	1	0	0	0	0	0	1

This portion of the Table only applies when LRT is considered

$\beta_D = 0.75$  when checking columns for maximum moment or maximum eccentricities and associated axial load;

and when Dead Load effects are of opposite sign to the net effects of other loads in a Group.

$\beta_D = 1.00$  when checking columns for maximum axial load and associated moment.

$\beta_E = 0.5$  for checking positive moments in rigid frames.

$\beta_E = 1.00$  for vertical earth pressure.

$\beta_E = 1.30$  for lateral earth pressure

$\beta_{wl} = 0.3$  for highway/bicycle/pedestrian loads only

$\beta_{wl} = 1.3$  for LRT+highway/bicycle/pedestrian loads only

D	Dead Load	T	Temperature
(L+I)H	Highway + Bicycle/pedestrian Live Load plus Impact	DT	Thermal Gradient
(L+I)P	Permit Load plus Impact	EQ	Earthquake
E	Earth Pressure	SF	Stream Flow Pressure
B	Buoyancy	PS	Prestress
W	Wind Load on Structure	$\alpha_{LRT}$	1 or 0 corresponding to presence or absence of LRT Train
WL	Wind on Live Load. 1.46 kN/m (100 lb/ft)	$L_{LRT}$	LRT Live Load
LF	Longitudinal Force from Live Load	$I_{LRT}$	Impact from LRT Live Load
CF	Centrifugal Force	CF <sub>LRT</sub>	Centrifugal Force from LRT Train
R	Rib Shortening	NF <sub>LRT</sub>	Nosing Force, LRT Train
S	Shrinkage	LF <sub>LRT</sub>	Longitudinal Force, LRT Train
		WL <sub>LRT</sub>	Wind on LRT Train
			BR
			RT <sub>2</sub>
			LT <sub>2</sub>
			DR
			Broken rail
			Radial thermal rail force
			Long. Restraint forces
			Derailment force, LRT

For suspension spans, applied loads shall be factored before stiffness analyses are performed. Linear superposition of element forces and stresses is not valid and shall not be used. Furthermore,  $\gamma$  x  $\beta_d$  is taken as 1.0 whenever it controls the design.

**3. CONCRETE - SEGMENTAL SUPERSTRUCTURE**

For segmental concrete superstructure, use 1989 AASHTO Guide Specifications for Construction and Design of Segmental Bridges, and the following:

**3.1 ALLOWABLE STRESSES**

Allowable Stresses shall follow the AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges, except as modified below for sand-lightweight concrete:

**3.1.1 Prestressing Steel**

Tensile stresses in prestressing tendons shall not exceed the following:

AASHTO Seg. 9.1

**3.1.1.1 Due to jacking force**

$$0.80 f'_s$$

**3.1.1.2 Immediately after tendon anchorage and prestress transfer**

BDS 9.15.1

At anchorages and couplers	0.70 $f'_s$
At internal tendon locations	0.83 $f'_s$
But not greater than	0.74 $f'_s$

**3.1.2 Prestressed Concrete**

AASHTO Seg 9.2

**3.1.2.1 Temporary stresses before losses at time of application of prestress**

**3.1.2.1.1 Compression**

Normal weight and Sand lightweight concrete	0.6 $f'_{ci}$
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AASHTO Seg 9.2.1.1

**3.1.2.1.2 Tension**

Longitudinal and transverse stresses in the precompressed tensile zone, *with* minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of 0.5  $f_{sv}$ ; internal tendons:

Normal weight concrete	0.25 $\sqrt{f'_{ci}}$ (MPa)
	[3 $\sqrt{f'_{ci}}$ (psi)]
Sand lightweight concrete	0.21 $\sqrt{f'_{ci}}$ (MPa)
	[0.85 x 3 $\sqrt{f'_{ci}}$ (psi)]

AASHTO Seg 9.2.1.2

Longitudinal and transverse stresses in the precompressed tensile zone, *without* minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of 0.5  $f_{sv}$ ; internal tendons:

Normal weight and Sand lightweight concrete	0 tension allowed
---	-------------------

Temporary tension in the precompressed tensile zone during construction, *with* minimum bonded auxiliary reinforcement through the possible crack zone to resist the tension:

Normal weight concrete	$0.50 \sqrt{f'_{ci}}$ (MPa) $[6 \sqrt{f'_{ci}}$ (psi)]	AASHTO Seg Table 7-2
Sand lightweight concrete	$0.42 \sqrt{f'_{ci}}$ (MPa) $[0.85 \times 6 \sqrt{f'_{ci}}$ (psi)]	
<b>3.1.2.2 Stresses at service level after losses - fully prestressed components</b>		
<b>3.1.2.2.1 Compression</b>		
Compressive stress due to effective prestress plus permanent (dead) loads shall not exceed:		
Normal weight and Sand lightweight concrete	$0.45 f'_c$	AASHTO Seg 9.2.2.1
<b>3.1.2.2.2 Tension in the precompressed tensile zone</b>		
Longitudinal and transverse stresses, <i>with</i> minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of $0.5 f_{sy}$ ; internal tendons:		
Normal weight concrete	$0.25 \sqrt{f'_c}$ (MPa) $[3 \sqrt{f'_c}$ (psi)]	AASHTO Seg 9.2.2.2
Sand lightweight concrete	$0.21 \sqrt{f'_c}$ (MPa) $[0.85 \times 3 \sqrt{f'_c}$ (psi)]	
Longitudinal and transverse stresses, <i>without</i> minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of $0.5 f_{sy}$ ; internal tendons:		
Normal weight and Sand lightweight concrete	0 tension allowed	
Under dead load only; either <i>with</i> or <i>without</i> minimum bonded auxiliary reinforcement through the joints:		
Normal weight and Sand lightweight concrete	0 tension allowed	BDS 9.15.2.2
<b>3.1.2.2.3 Tension in other areas</b>		
Bonded reinforcement shall be provided to carry the calculated tensile force at a stress of $0.5 f_{sy}$ :		
Normal weight concrete	$0.50 \sqrt{f'_c}$ (MPa) $[6 \sqrt{f'_c}$ (psi)]	AASHTO Seg. 9.2.2.4
Sand lightweight concrete	$0.42 \sqrt{f'_c}$ (MPa) $[0.85 \times 6 \sqrt{f'_c}$ (psi)]	
<b>3.1.2.3 Cracking stress</b>		
Modulus of rupture:		
Normal weight concrete	$0.63 \sqrt{f'_c}$ (MPa) $[7.5 \sqrt{f'_c}$ (psi)]	BDS 9.15.2.3

Sand lightweight concrete	$0.52 \sqrt{f'_c}$ (MPa) $[0.85 \times 7.5 \sqrt{f'_c}$ (psi)]	
<b>3.1.2.4 Bearing stress</b>		
Post-tensioned anchorage at service level	21 MPa (3000 psi)	BDS 9.15.2.4
<b>3.1.2.5 Shear</b>		
Shear design shall follow the AASHTO LRFD criteria based on modified compression field theory.		AASHTO LRFD 1993
<b>3.1.2.6 Web Shear</b>		
Principle tensile stress limits during service conditions shall not exceed the following limits:		
Normal weight concrete	$0.25 \sqrt{f'_c}$ (MPa) $[3.0 \sqrt{f'_c}$ (psi)]	
Sand lightweight concrete	$0.21 \sqrt{f'_c}$ (MPa) $[0.85 \times 3.0 \sqrt{f'_c}$ (psi)]	
<b>3.2 STRENGTH REDUCTION FACTORS</b>		
Superstructure Strength Reduction Factors, $\phi$ , shall be taken from the 1989 Guide Specifications for Design and Construction of Segmental Concrete Bridges, and amended as follows:		

**3.2.1 Normal and Sand lightweight concrete**

Fully bonded tendons, Type A joint

Flexure  $\phi_f = 0.95$

Shear and torsion  $\phi_v = 0.85$

Unbonded tendons, Type A joint

Flexure  $\phi_f = 0.90$

Shear and torsion  $\phi_v = 0.80$

**3.2.2 Bearing**

For anchorage zones:

Normal weight concrete  $\phi_b = 0.85$

Sand lightweight concrete  $\phi_b = 0.70$

**3.3 MATERIALS**

For segmental concrete, use 1999 AASHTO Guide Specifications for Construction and Design of Segmental Bridges, and the following:

**3.3.1 Concrete**

**3.3.1.1 Normal weight concrete**

56-day minimum strength  $f_c = 55$  MPa (8000 psi)  
 at time of stressing  $f_c = 25$  MPa (3600 psi)

**3.3.1.2 Sand-lightweight concrete**

56-day minimum strength  $f_c = 55$  MPa (8000 psi)  
 at time of stressing  $f_c = 25$  MPa (3600 psi)

**3.3.1.3 Creep and shrinkage**

The specific creep coefficient, as determined in accordance with ASTM C 512, after 365 days of loading, shall not exceed 75 millionths/MPa.

The ultimate creep coefficient for the bridge conditions, loading at 28 days shall be  $\phi_c = 2.35$

The shrinkage strain of Portland cement concrete shall not exceed 0.045% after 180 days of drying in accordance with ASTM C 157.

The ultimate shrinkage strain for bridge conditions shall be  $\epsilon_{ult} = 224 \times 10^{-6}$  mm/mm (in/in)

**3.3.2 Prestressing Steel**

**3.3.2.1 Strand Prestressing Steel**

15 mm or 13 mm AASHTO M203 (ASTM A416), uncoated seven-wire stress-relieved or low relaxation strand Grade 270.

**3.3.2.1.1 Properties**

Guaranteed Ultimate Tensile strength  $f_s = 1860$  MPa (270 ksi)  
 Yield strength  $f_v^* = 1674$  MPa (243 ksi)

Modulus of Elasticity	E= 197,000 MPa	(28,500 ksi)
Anchor set	$\Delta = 6$ mm	(1/4 inch)
<b>3.3.2.1.2 Prestress losses</b>		
Standard galvanized steel ducts:		
Wobble coefficient	$\kappa = 0.0007$ /m	(0.0002 /ft)
Friction coefficient	$\mu = 0.2$ /rad	
Deviated polyethylene duct (external to concrete):		
Wobble coefficient	$\kappa = 0.0007$ /m	(0.0002 /ft)
Friction coefficient	$\mu = 0.25$ /rad	
Prestressing steel may be bonded, internal to the section, or unbonded, external to the section.		
<b>3.3.2.2 Bar Prestressing Steel</b>		
ASTM A722 (Type II) high strength threaded bars. Standard galvanized steel ducts.		
Tensile strength	$f_s = 1030$ MPa	(150 ksi)
Modulus of Elasticity	E= 207,000 MPa	(30,000 ksi)
Anchor set	$\Delta = 2$ mm	(1/16 inch)
Friction coefficient	$\mu = 0.30$ /rad	
Wobble coefficient	$\kappa = 0.0007$ /m	(0.0002 /ft)
<b>3.3.3 Mild Steel Reinforcement</b>		
All mild steel reinforcement shall be ASTM A706 (Grade 60). The following properties shall be used in the design:		
Minimum yield strength	$f_y = 415$ MPa	(60 ksi)
Maximum tensile stress	$f_u = 738$ MPa	(107 ksi)
Modulus of elasticity	$E_s = 200,000$ MPa	(29,000 ksi)
 <b>4. CONCRETE – CAST IN PLACE SUPERSTRUCTURE &amp; SUBSTRUCTURE</b>		
For cast-in-place construction, use Caltrans Bridge Design Specification Manual (BDS), except as modified below.		
<b>4.1 ALLOWABLE STRESSES</b>		
BDS shall apply, except as modified below:		
<b>4.1.1 Concrete</b>		
<b>4.1.1.1 Joint and Web Shear</b>		
<b>4.1.1.1.1 Principle tensile stress limits for Functional Evaluation Earthquake (FEE), or without special reinforcement:</b>		
Normal weight concrete		$0.29 \sqrt{f'_c}$ (MPa)
		$[3.5 \sqrt{f'_c}]$ (psi)

4.1.1.1.2 Principle tensile stress limits for Safety Evaluation Earthquake (SEE):

Normal weight concrete	$0.42 \sqrt{f'_c}$ (MPa)
	$[5 \sqrt{f'_c}$ (psi)]

4.1.1.1.3 Principal compression stress limits for SEE:

Normal weight concrete	$0.30 f'_c$
------------------------	-------------

**4.2 STRENGTH REDUCTION FACTORS**

Except for the following, BDS shall apply.

**4.2.1 Substructure**

Substructure shall be designed using Load Factor Design, with Strength Reduction Factors based on Caltrans BDS 8.16.1.2.2 except for Seismic Load Combinations. See Section 7.9 of this Criteria.

**4.3 MATERIALS**

Except for the following, BDS shall apply.

**4.3.1 Concrete**

**4.3.1.1 Superstructure**

**Normal weight concrete**

56-day minimum strength	$f'_c = 55$ MPa	(8000 psi)
at time of stressing	$f'_c = 25$ MPa	(3600 psi)

**Light weight concrete for panels and ribs**

28-day minimum strength	$f'_c = 45$ MPa	(6500 psi)
at time of stressing	$f'_c = 25$ MPa	(3600 psi)

**4.3.1.2 CISS piling**

28-day minimum strength	$f'_c = 25$ MPa	(3600 psi)
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**4.3.1.3 Piers**

28-day minimum strength	$f'_c = 30$ MPa	(4350 psi)
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**4.3.2 Prestressing Steel**

**4.3.2.1 Strand Prestressing Steel**

15 mm AASHTO M203 (ASTM A416), uncoated seven-wire stress-relieved or low relaxation strand Grade 270.

**4.3.2.1.1 Properties**

Guaranteed Ultimate Tensile strength	$f'_s = 1860$ MPa	(270 ksi)
Yield strength	$f^*_y = 1674$ MPa	(243 ksi)
Modulus of elasticity	$E = 197,000$ MPa	(28,500 ksi)
Anchor set	$\Delta = 6$ mm	(1/4 inch)

4.3.2.1.2 Prestress losses

For conventional CIP Concrete on Falsework:

Standard galvanized steel ducts:

Wobble coefficient  $\kappa = 0.0007$  (/m) (0.0002 /ft)  
 Friction coefficient  $\mu = 0.20$  (/rad)

4.3.2.1.3 Bar Prestressing Steel

ASTM A722 (Type II) high strength threaded bars. Standard galvanized steel ducts.

Tensile strength  $f'_s = 1030$  MPa (150 ksi)  
 Modulus of Elasticity  $E = 207,000$  MPa (30,000 ksi)  
 Anchor set  $\Delta = 2$  mm (1/16 inch)  
 Friction coefficient  $\mu = 0.30$  /rad  
 Wobble coefficient  $\kappa = 0.0007$  /m (0.0002 /ft)

**5. CORROSION PROTECTION**

**5.1 PILES**

The pile shells shall be designed for corrosion to ensure that the minimum design thickness will be present at the end of the 150-year design life. The minimum sacrificial steel corrosion allowance to be added to the minimum structural design thickness of the shell is specified below. Applied corrosion protection coatings shall not be used.

The pile will be protected differently in four zones, as defined below:

Piers	Depth	Top	Bottom	Corr. Allow.
E3-6	Exposed Zone	Top of Pile	1 m below scour depth	20 mm
E3-6	Deep buried zone	1 m below scour depth	Pile tip	0 mm
E7-16	Top buried zone	Top of Pile	1 m below pile cap	3 mm
E7-16	Deep buried zone	1 m below pile cap	Pile tip	0 mm

The design depths of the corrosion allowances shall take account of the under-drive acceptance criteria defined in the Piling Special Specification Provisions to ensure that the necessary design thickness is maintained. The outside diameter of the pile, including the steel corrosion allowance, shall be constant. The section of the pile embedded in the pile cap shall have the same steel corrosion allowance as the exposed upper section of the pile. No corrosion allowance shall be added to the shear connectors. The steel reinforcing bars inside the pile shall be uncoated.

**5.2 REINFORCEMENT PROTECTION**

Splash Zone (Piers in water): Epoxy-coated reinforcement shall be used from the top of the splash zone to the bottom of the footing, per SSP 52M\_PURP (Purple epoxy). This shall include the first exterior mats of top, bottom, and side pile cap reinforcement.

**5.3 BARRIERS**

Epoxy-coated reinforcement shall be used in all barrier railings.

**6. STRUCTURAL STEEL**

**6.1 MATERIALS**

Unless modified herein, or specified in the BDS, structural steel shall comply with the AASHTO Materials Specifications, and ASTM.

**6.1.1 Structural Steel**

ASTM A709 Gr. 50, AASHTO M270, and A709HPS Gr. 70W

**6.1.2 Miscellaneous Structural Steel**

High Strength Bolts	ASTM A325
Dowels	ASTM A36

**6.1.3 Orthotropic Deck**

Deck plate thickness, min.	16 mm	(5/8 inch)
Rib plate thickness, min.	8 mm	(5/16 inch)
Weld between deck plate and ribs	80% penetration weld	
Epoxy asphalt overlay (or equivalent) thickness	50 mm	(2 inch)
Deflection limitations:		
Deflection of deck plate	Span length /300	
Deflection of ribs	Span length /1000	
Relative live load deflection between adjacent ribs	2.5 mm	(0.1 inch)

**6.1.4 Expansion Joints**

Bridge expansion joints shall be designed for movement demands from temperature rise and fall, creep and shrinkage, functional evaluation earthquake and safety evaluation earthquake. Temperature movements will not be combined with seismic movements. Creep and shrinkage movements shall be increased by 50%.

The modular joint seal assemblies shall be designed in accordance with the following criteria:

- Service  
 $0 \leq r \leq 75mm$   
where r is the unsupported length of a strip seal supported by adjacent beams
- Functional Evaluation Earthquake  
 $0 \leq r \leq 130mm$
- Safety Evaluation Earthquake  
Closing:  $0 \leq r$   
Opening: Support bar shall not be unseated

Bike path expansion joints shall be designed for movement demands from temperature rise and fall and creep and shrinkage.

**7. SEISMIC DESIGN**

Seismic design shall be performed in accordance with BDS, modified by or augmented with pertinent provisions of ATC-32 and project specific criteria as detailed in this document.

**7.1 SEISMIC LOADING**

**7.1.1 Design Seismic Loading**

The initial design shall be based on using the results of a linear dynamic response spectrum analysis for the 3-dimensional response spectrum loading at 5% damping shown in the following table:

E3-6			E7-16			E3-16	
Period	FN	FP	Period	FN	FP	Period	Vertical
(sec)	ACC(g)	ACC(g)	(sec)	ACC(g)	ACC(g)	(sec)	ACC(g)
0.000	0.747	0.747	0.000	0.497	0.497	0.000	0.340
0.200	1.134	1.134	0.200	0.801	0.801	0.100	0.800
0.300	1.594	1.594	0.300	1.035	1.041	0.120	0.820
0.400	1.860	1.860	0.400	1.346	1.343	0.200	0.770
0.500	1.860	1.860	0.500	1.468	1.471	0.400	0.700
0.600	1.796	1.796	0.600	1.521	1.521	0.600	0.600
0.800	1.613	1.613	0.800	1.376	1.376	0.800	0.520
1.000	1.453	1.453	1.000	1.205	1.207	1.000	0.440
1.500	1.106	1.106	1.500	0.950	0.952	1.500	0.310
2.000	0.870	0.830	2.000	0.793	0.793	2.000	0.230
2.500	0.611	0.551	2.500	0.601	0.589	3.000	0.160
3.000	0.462	0.397	3.000	0.483	0.465	4.000	0.100
3.500	0.353	0.292	3.500	0.368	0.339	5.000	0.800
4.000	0.280	0.223	4.000	0.292	0.257		
4.500	0.228	0.180	4.500	0.244	0.202		
5.000	0.191	0.149	5.000	0.209	0.163		
5.500	0.158	0.124	5.500	0.174	0.135		
6.000	0.132	0.105	6.000	0.147	0.114		

The final design shall be based on using the results of the nonlinear dynamic analysis for three sets of spectrum-compatible tri-component multiple-support time histories for, both an event on the Hayward fault and an event on the San Andreas fault. The maximum response from the six sets of ground motions shall be used for design. The design shall be verified using static pushover analyses to target displacements established by response spectrum analysis.

**7.1.2 Seismic Loading During Construction**

The bridge shall be designed to resist an equivalent static load of 0.1g for configurations occurring under the assumed construction sequence.

**7.2 PERFORMANCE CRITERIA**

The desired performance may be defined in terms of two levels of earthquake:

- A functional evaluation earthquake
- A safety evaluation earthquake

**7.2.1 Functional Evaluation Earthquake**

After a functional evaluation earthquake the bridge may suffer "some to no loss of operation," with minor damage to structure.

SDCB

Minor damage implies essentially elastic performance, and is characterized by:

- Minor inelastic response
- Narrow cracking in concrete
- No apparent permanent deformations
- Damage to expansion joints that can be temporarily bridged with steel plates

**7.2.2 Safety Evaluation Earthquake**

After a safety evaluation earthquake the damage to the bridge may be described as "minor to moderate damage with some loss of operation."

SDCB

Moderate damage implies visible and significant signs of damage with repairs or stabilization likely to be completed under emergency contracts and is characterized by:

- Minimal damage to the superstructure
- Limited damage to piers, including yielding of reinforcement and spalling of concrete cover
- Minimal damage to Piles and Pile caps
- Small permanent deformations, not interfering with serviceability of the bridge
- Damage to expansion joints that can be temporarily bridged with steel plates

**7.2.3 Limited Ductility Structure**

The bridge shall be designed as a limited ductility structure as defined in ATC-32 with stable response to ground motions equivalent to the safety evaluation earthquake. The stability of the structure shall be demonstrated by means of pushover analysis or an equivalent method of structural evaluation. This means:

- The bridge shall have a clearly defined inelastic mechanism for response to lateral loads.
- Inelastic behavior shall be restricted to piers and hinge beam fuses.
- The detailing and proportioning requirements for full-ductility structures as defined in ATC-32 shall be met.

**7.2.4 Collapse Avoidance**

For seismic loading during construction, the performance objective shall be to avoid collapse.

**7.3 SEISMIC ANALYSIS**

**7.3.1 Skyway Structure**

The preliminary design of the Skyway structure may be performed using response spectrum analysis. Response spectrum analysis shall be used to establish target displacements for non-linear static (i.e. pushover) analysis. A displacement-based design approach as outlined in Section 7.3.3 shall be the basis for the seismic design of the skyway structure.

**7.3.2 Force-Based Design**

Force-based design may be used for the preliminary sizing of the Skyway structure. The piers may be designed with a Z-factor of 3.0, assuming that  $T/T^* \geq 1$ , where  $T^*$  corresponds to the peak of the input energy spectrum. A Z-factor of  $1\frac{1}{2}$  may be used for piles.

ATC-32 3.21.10.1,  
ATC-32 3.21.11

ATC-32 Fig. R3-13

**7.3.3 Deformation-Based Design**

Deformation-based design shall include direct design using non-linear time history analysis or design verification using non-linear static (i.e. pushover) analysis. All deformation-based design shall use models that reflect the "best estimate" of the likely structure and soil condition at the time of the safety evaluation earthquake.

When non-linear time history analysis is used as the basis for design, deformation demands shall be the maximum instantaneous demands for any of the six 3-dimensional input motions.

When pushover analysis is used to calculate the deformation capacity of the structures, the deformation capacity, corresponding to the limiting strains in materials, shall exceed or be equal to the calculated deformation demand. For a portion of the structure that can be characterized as a single-degree-of-freedom system, pushover shall be to a target displacement obtained from elastic response spectrum analysis results.

The deformation and/or displacement capacity of structures shall be evaluated using the limiting material strains and conditions contained in Section 7.12 of this document.

**7.3.4 Soil-Structure Interaction Effects**

Soil-structure interaction effects shall be considered in all analyses. Foundation dynamic characteristics shall be incorporated into the analysis with discrete elements representing piles and footings with appropriate representation of the effects of soil structure interaction (SSI). Depth varying free-field motions shall be applied to the analysis model along the buried length of the appropriately discretized piles.

SDCB

**7.4 COMBINATION OF EFFECTS**

For response spectrum analysis, seismic effects from excitation in three orthogonal directions shall be combined by the 30% rule, where the forces resulting from excitation in one direction are combined with 30% of the forces resulting from excitation in the orthogonal directions.

ATC-32 3.21.9.2

**7.5 PERMANENT DISPLACEMENTS**

Maximum permanent displacement demands following the safety evaluation earthquake shall be obtained directly from non-linear time history analyses for the six 3-dimensional input motions.

The estimated permanent displacement of the bridge, as established by nonlinear dynamic analysis, shall not exceed 300 mm (12 inches) at the deck level.

To ensure the structural integrity and serviceability of the bridge after the safety evaluation earthquake, permanent vertical settlements at the pile caps shall not exceed 50 mm (2 inches)

ATC 32 3.21.10.3

**7.6 CAPACITY DESIGN**

Structural elements, other than columns constituting the ductile lateral load resisting part of the structure, shall be designed to resist the over-strength capacity of the ductile elements.

ATC-32 3.21.14, 10.19.3,  
10.20, 10.24.8

**7.7 HINGE BEAMS**

Hinge beams frames shall be proportioned for the forces and displacements calculated by the appropriate non-linear dynamic time history analysis of the connected frames.

ATC-32 3.21.12, 10.29.8

**7.8 P-Δ EFFECTS**

P- Δ effects in Piers maybe ignored when:

$$W\delta_u \leq 0.25V_0H$$

where:

W = weight on the pier

$\delta_u$  = maximum drift at the top of the pier based on elastic response spectrum analysis

$V_0$  = base shear strength of the pier

H = height of the pier

ATC-32 3.21.15

**7.9 STRENGTH REDUCTION FACTORS**

Strength reduction factors shall be per ATC-32.

ATC-32 8.16.1.2.2

**7.10 MATERIAL PROPERTIES FOR DUCTILE ELEMENTS**

**7.10.1 Design flexural strength**

The design flexural strength of plastic hinges shall be based on expected material strengths:

$$f_{ce} = 1.3 f_c$$
$$f_{ye} = 1.1 f_y$$

ATC-32 8.16.2.4.1

Maximum concrete strains at the design flexural strength shall not exceed 0.004. If moment curvature analysis is used to determine the design flexural strength, the steel strains shall be limited to 0.015.

**7.10.2 Maximum plastic moment**

The maximum plastic moment shall be calculated using one of the following two methods.

ATC-32 8.16.4.4, 10.63

1. The maximum plastic moment of sections shall be based on maximum feasible material strengths:

$$f_{co} = 1.7 f'_c$$
$$f_{yo} = 1.25 f_y$$

Maximum plastic moments shall be determined from moment-curvature analysis that considers the effects of concrete confinement and strain hardening of the reinforcement. The maximum plastic moment shall be the calculated moment at the design deformation of the element.

2. The maximum plastic moment of sections shall be based on expected material strengths:

$$f_{co} = 1.3 f'_c$$
$$f_{yo} = 1.1 f_y$$

Maximum plastic moments shall be determined from moment-curvature analysis that considers the effects of concrete confinement and strain hardening of the reinforcement. The maximum plastic moment shall be 1.15 times the calculated moment at the design deformation of the element.

**7.11 DEFORMATION CAPACITY**

The deformation capacity of structures shall be calculated using plastic hinge lengths calculated according to ATC-32 8.18.2.4.2 and rotational capacities corresponding to the allowable material strains from Section 7.12.

For the piers, the allowable material strains shall be assumed to be the average extreme fiber strains over the plastic hinge length. For the piles, the allowable strains shall be assumed to be the maximum extreme fiber strains anywhere within the plastic hinge.

Plastic hinge lengths for hollow reinforced concrete sections and pile sections shall be verified by laboratory testing and/or detailed non-linear analysis.

**7.12 ALLOWABLE STRAINS**

**7.12.1 Normal weight concrete**

Allowable strains in normal weight concrete shall be:

Piers (Average extreme fiber strains in plastic hinge):

Functional evaluation earthquake

$$\epsilon_c^{Func} = 0.004$$

Safety evaluation earthquake

$$\epsilon_c^{Safety} = 2/3 \epsilon_{cu}$$

where  $\epsilon_{cu}$  is the ultimate concrete strain according to the Mander model

(Mander et.al. J. Struct. Engineering, ASCE, 1988 114(8), 0 1804 – 1849)

Piles (Maximum extreme fiber strains in potential plastic hinge):

Functional evaluation earthquake

$$\epsilon_c^{Func} = 0.004$$

Safety evaluation earthquake

$$\epsilon_c^{Safety} = 0.01$$

**7.12.2 Reinforcing Steel**

Allowable strains in reinforcing steel shall be:

Piers (Average extreme fiber strains in plastic hinge):

Functional evaluation earthquake

$$\epsilon_s^{Func} = 0.015$$

Safety evaluation earthquake

$$\epsilon_s^{Safety} = 1/2 \epsilon_{su}$$

Where  $\epsilon_{su}$  is the steel strain at ultimate stress. For Grade 60 (A706)

reinforcement,  $\epsilon_{su}$  may be taken as:

Confinement bars No. 10 – 25 (No. 3 – 8)

$$\epsilon_{su} = 0.12$$

Main bars No. 29 – 57 (No. 9 – 18)

$$\epsilon_{su} = 0.09$$

Piles (Maximum extreme fiber strains in potential plastic hinge):

Functional evaluation earthquake

$$\epsilon_s^{Func} = 0.015$$

Safety evaluation earthquake

$$\epsilon_s^{Safety} = 0.02$$

Where  $\epsilon_{su}$  is the steel strain at ultimate stress. For Grade 60 (A706)

reinforcement,  $\epsilon_{su}$  may be taken as:

Confinement bars No. 10 – 25 (No. 3 – 8)

$$\epsilon_{su} = 0.12$$

Main bars No. 29 – 57 (No. 9 – 18)

$$\epsilon_{su} = 0.09$$

Grade 50 hardening strains may be taken as:

Bars No. 10 – 25 (No. 3 – 8)

$$\epsilon_{sh} = 0.0150$$

Bars No. 29 – 36 (No. 9 – 11)

$$\epsilon_{sh} = 0.0100$$

ATC-32 C3.21.11.1

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	Bar No. 43 (No. 14)	$\epsilon_{sh} = 0.0075$	
	Bar No. 57 (No. 18)	$\epsilon_{sh} = 0.0050$	
<b>7.12.3</b>	<b><i>Structural Steel Pile Shells (Casings)</i></b>		
	Functional evaluation earthquake	$\epsilon_s^{Func} = 0.015$	
	Safety evaluation earthquake	$\epsilon_s^{Safety} = 0.02$	
<b>7.13</b>	<b>DESIGN FOR JOINT SHEAR</b>		
	Superstructure/pier joints, pier/footing joints, and other joints between flexural members shall be proportioned and reinforced for joint shear in accordance with section 8.34 of ATC-32.		ATC 32 8.34
<b>7.14</b>	<b>COLUMN SHEAR DESIGN</b>		
	Shear demand shall be the maximum plastic shear obtained using the maximum plastic moment capacity of the column. Shear capacity shall be determined according to Section 3.6 of Version 1.1 of the Caltrans Seismic Design Criteria.		
<b>8.</b>	<b>GEOTECHNICAL AND FOUNDATION DESIGN</b>		
	The design of the foundations will be based on the Foundation Reports from Fugro and Earth Mechanics.		



**SAN FRANCISCO - OAKLAND BAY BRIDGE  
EAST SPAN SEISMIC SAFETY PROJECT**

**STRUCTURAL DESIGN CRITERIA  
YBI & OAKLAND APPROACH STRUCTURES**

Prepared by:

Moffatt & Nichol Engineers

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**1. GENERAL**

The bridge shall be designed in accordance with "Caltrans Bridge Design Specifications Manual (1995) (BDS)," modified or augmented as detailed in this design document.

In addition to bridge and site specific criteria, pertinent sections of the following standards or codes have been employed for such modifications or augmentations.

- "AASHTO Standard Specifications for Highway Bridges", 16<sup>th</sup> Edition, 1992
- "AASHTO LRFD Specifications for Highway Bridge Design" 2<sup>nd</sup> Edition, 1998, AASHTO, Washington, DC, 1994
- "Sacramento Light Rail Transit Design Criteria", May 1993
- "San Francisco-Oakland Bay Bridge East Span Seismic Safety Project Light Rail Transit Design Criteria", 1999
- "Improved Seismic Design Criteria for California Bridges: Provisional Recommendations" ATC-32 Report, June 30, 1996
- "Caltrans Seismic Design Criteria", Version 1.2, November 2001
- "AISC Manual of Steel Construction Load & Resistance Factor Design" (LRFD), 1999 Edition
- "AISC Manual of Steel Construction LRFD", 3<sup>rd</sup> Edition, 2001
- Guide to Stability Design Criteria for Metal Structures, 5<sup>th</sup> Edition, Structural Stability Research Council
- Seismic Ground Motion for SFOBB, East Span Seismic Safety Project, Furgo, Earth Mechanics, JV Report
- Draft Geotechnical Foundation Report for the Yerba Buena Island (YBI) Approach and Main Span Structures, July 2001, by Fugro-Earth Mechanics Joint Venture
- Bridge Welding Code, AWS D1.5M, 1996
- Structural Welding Code - Steel AWS D1.1, 1998
- SFOBB East Span-Seismic Design Criteria Basis, June 27, 2000

"BDS"

"AASHTO"

"AASHTO LRFD"

"Sac. LRT"

"SFOBB LRT"

"ATC-32" Reports

SDC

"AISC LRFD"

"AISC"

"SSRC"

"Seismic Ground Motion" Report

"Foundation Report"

"AWS D1.5"

"AWS D1.1"

"SDCB"

**2. DESIGN LOADS**

This section covers all design loads except for seismic demands discussed in Section 7.

**2.1 STRUCTURAL DEAD LOADS -DL**

Unless specified herein or in BDS, all dead loads shall be as specified in the Specifications cited in Article 1, General, of BDS.

**2.2 OTHER PERMANENT LOADS -SDL**

Other permanent loads are assumed to be applied at the time of construction except for utilities and dead load of Light Rail Transit (LRT) appurtenances.

**2.2.1 Vehicular and Pedestrian Barriers**

Concrete roadway barrier	
Modified 732 barrier	8.5 kN/m (585 lb/ft)
Temporary K Rail	5.7 kN/m (390 lb/ft)
Steel pedestrian handrail	0.60 kN/m (41 lb/ft)

**2.2.2 Wearing Surface**

Transition structures, Oakland Approach Structures and Eastbound On-Ramp (Future)	1.675 kN/m <sup>2</sup> (35 lb/ft <sup>2</sup> )
Temporary Detour Structures & Viaduct	None
Bicycle/pedestrian path (concrete)	None

**2.2.3 Provision for Utilities:**

Transition structures & Oakland Approach Structures	12.6 kN/m (869 lb/ft)
Temporary Detour Structures	7.2 kN/m (500 lb/ft)

This number includes allowance for miscellaneous metal, drainage, lighting system, utilities, utility supports, service platform utilities, and service platform support.

**2.3 LIVE LOADS - LL+I**

Live loads shall be considered for two roadway configurations:

- Highway traffic (6 lanes) on the Westbound Structure and upper level of the Viaduct; highway traffic (6 lanes) plus a bicycle/pedestrian facility on the south side of the Eastbound Structure.
- An LRT system on the inside lane plus highway traffic (4 lanes) on the Westbound Structure; and LRT system on the inside lane plus highway traffic (4 lanes) on the Eastbound Structure with a bicycle/pedestrian facility on the south side.
- Highway traffic (5 lanes) on the Westbound and Eastbound Temporary Detour Structures.

**2.3.1 Standard Truck and Lane Loads**

Vehicular live load shall be AASHTO HS20-44

Highway truck loads shall be positioned both transversely and longitudinally so as to produce the maximum influence on the structure. LRT loads shall be applied on the inside lane of each transition structure and positioned longitudinally to produce the maximum influence.

**2.3.2 Load reduction factors for multiple lane loading**

Load reduction factors for multiple lane loading shall be applied to highway lanes only. Full LRT Train Loading shall be combined with highway traffic loading to produce the most critical stress condition in the member.

**2.3.3 Permit Vehicle Loads**

Permit loads (P13) shall be considered for design of the Transition structures, Oakland Approach Structures and Eastbound On-Ramp only.

BDS 3.7.7, Figure 3.7.7A,  
 BDS Figure 3.7.7B and  
 3.11.4.3

**2.3.4 Pedestrian Loads**

Pedestrian loading shall be in accordance with BDS.

**2.3.5 Maintenance Vehicle**

In the bike path, one pick-up truck, including impact shall be used for the deck design.

Pick-up (Gross Weight) H-4 (8000 lb)

**2.3.6 Live Load Contribution Under Seismic Conditions**

A reduced factored live load of 8.75 kN/m (600 lb/ft) acting over the full width of the Transition Structures and Oakland Approach Structures, shall be considered for combination with seismic demands.

**2.3.7 Live Load on Platforms and Landings**

**2.3.7.1 General**

Minimum 4.0 kN/m<sup>2</sup> (85 psf)

**2.3.7.2 Utility Platform**

For live load on utility platform, see Mechanical Design Criteria

**2.4 THERMAL EFFECTS - T**

BDS 3.16

**2.4.1 Uniform Temperature -  $T_c$  and  $T_s$**

Design temperature range shall correspond to BDS requirements for coastal areas:

Mean Temperature	27°C (81°F)
Rise or Fall	
Concrete	17°C (30°F)
Steel	22°C (40°F)

**2.4.2 Coefficient of thermal expansion**

Normal weight Concrete	10.8x10 <sup>-6</sup> /°C (6.0x10 <sup>-6</sup> /°F)
Steel	11.7x10 <sup>-6</sup> /°C (6.5x10 <sup>-6</sup> /°F)

**2.5 WIND-W**

The YBI structures and the Oakland Approach Structures shall be designed for wind loads per BDS as a minimum.

BDS 3.15

**2.6 COMBINATION OF LOADS**

**2.6.1 Service and Ultimate Limit States**

Loading combinations shall be in accordance with Caltrans BDS Tables 3.22.1A and 3.22.1B, and "Sacramento Regional Transit District Light Rail Design Criteria".

BDS, Sac. LRT

**3. CONCRETE - CAST IN PLACE SUPERSTRUCTURE & SUBSTRUCTURE**

For cast-in-place construction, use BDS, except as modified below:

**3.0.1 Prestressing Steel**

**3.1 Strand Prestressing Steel**

### 3.1.1 Prestress losses

For conventional CIP Concrete on Falsework:

Standard galvanized steel ducts:

Wobble coefficient  $\kappa=0.0007(/m)$  (0.0002/ft)

Friction coefficient  $\mu=0.20(/rad)$

## 4. REINFORCEMENT PROTECTION

Epoxy-coated reinforcement (green) shall only be used in the following components:

- 1) Concrete barriers on all structures except Temporary Detours.
- 2) Concrete deck overlay on the Viaduct.
- 3) Diagonal bars in hinges.

## 5. STRUCTURAL STEEL FOR TEMPORARY DETOUR STRUCTURES

### 5.1 MATERIALS

Unless modified herein, or specified in the BDS, structural steel shall comply with the AASHTO Materials Specifications, and ASTM.

#### 5.1.1 Structural steel:

Plates: ASTM A709M Grade 345

Rolled Shapes - ASTM A709M Grade 345

Structural Pipe - ASTM A106 Grade C ( $F_y=275$  MPa)

#### 5.1.2 Fasteners:

ASTM A325, high strength bolts unless noted otherwise

#### 5.1.3 Anchor bolts:

ASTM 354 Grade BD

### 5.2 WELDING:

Welding shall comply with the latest version of ANSI/AASHTO/AWS D1.5 (Non-tubular members), D1.4 (Reinforcement) and D1.1 (Tubular members).

### 5.3 FATIGUE:

Components shall be checked per BDS to withstand fatigue induced by 500,000 cycles of loading.

BDS

## 6.0 EXPANSION JOINTS AND SEAT WIDTHS

### 6.1 MODULAR EXPANSION JOINTS

Bridge modular expansion joints shall be designed for movement demands from temperature rise and fall, creep and shrinkage, functional evaluation earthquake and safety evaluation earthquake. Temperature movements shall not be combined with seismic movements.

Modular expansion joints shall be sized to be fully functional throughout the range of movement associated with the FEE event. During the opening cycle of the SEE event, the joint may open beyond its normal operating range resulting in parting or tearing of the flexible seals. However, the length of the support bars shall be sized such that separation beams and bearing bars will not fall through the open gap in the deck.

**6.2 HINGE AND ABUTMENT SEAT WIDTHS**

Seat widths shall be sized such that joints can open fully under the SEE event without losing seat on the bearings. As a redundancy measure, a nominal number of longitudinal restrainer cables shall be provided at the hinges; cable lengths and connection details shall be arranged such that the cables offer little resistance up to the point of full SEE gap opening but reach full capacity prior to unseating.

**6.3 BIKE PATH AND TEMPORARY DETOUR STRUCTURES EXPANSION JOINTS**

Bike path and Temporary Detour Structures expansion joints shall be designed for movement demands from temperature rise and fall and creep and shrinkage.

**7. SEISMIC DESIGN**

Seismic design shall be performed in accordance with BDS, modified by or augmented with pertinent provisions of SDC and project specific criteria as detailed in this document.

**7.1 SEISMIC LOADING**

**7.1.1 Design Seismic Loading - Permanent Structures**

**7.1.1.1 Response Spectrum Analysis**

The design shall be based on using the results of a linear dynamic response spectrum analysis for the 3-dimensional response spectrum loading at 5% damping shown in the following tables:

YBI Structures:

Safety Evaluation Earthquake(SEE)			
Period (Second)	Acceleration (g)		
	Fault Normal	Fault Parallel	Vertical
0.010	0.627	0.627	0.646
0.020	0.627	0.627	0.646
0.030	0.627	0.627	0.646
0.050	0.820	0.820	1.038
0.075	1.036	1.036	1.255
0.100	1.236	1.236	1.304
0.120	1.360	1.360	1.290
0.150	1.546	1.546	1.262
0.170	1.591	1.591	1.198
0.200	1.658	1.658	1.123
0.240	1.654	1.654	1.034
0.250	1.653	1.653	1.017
0.300	1.626	1.626	0.935
0.400	1.480	1.480	0.799
0.500	1.277	1.277	0.675
0.750	1.008	0.978	0.550
1.000	0.794	0.755	0.441
1.500	0.576	0.508	0.312
2.000	0.444	0.373	0.235
3.000	0.270	0.220	0.156
4.000	0.186	0.148	0.106
5.000	0.140	0.108	0.079
6.000	0.105	0.080	0.059
7.000	0.081	0.063	0.047
7.500	0.072	0.055	0.043
8.000	0.065	0.048	0.037
9.000	0.053	0.039	0.030
10.000	0.045	0.032	0.024

Functional Evaluation Earthquake (FEE)	
Period (Second)	Acceleration (g)
0.010	0.300
0.020	0.300
0.030	0.311
0.040	0.361
0.050	0.402
0.060	0.445
0.070	0.476
0.080	0.508
0.090	0.541
0.100	0.570
0.120	0.613
0.140	0.653
0.150	0.669
0.170	0.682
0.200	0.686
0.240	0.654
0.270	0.631
0.300	0.606
0.400	0.509
0.500	0.416
0.600	0.349
0.700	0.295
0.800	0.251
0.900	0.223
1.000	0.194
1.200	0.161
1.300	0.144
1.500	0.117
1.800	0.093
2.000	0.078
3.000	0.042

#### 7.1.1.2 Time History

The design shall be checked using the results of the nonlinear dynamic analysis for three sets of the spectrum-compatible tri-component multiple-support time histories for, both an event on the Hayward fault and an event on the San Andreas Fault. The maximum response from the six sets of ground motions shall be used for design. The design shall be verified using static pushover analyses to target displacements established by response spectrum analysis.

#### 7.1.2 Design Seismic Loading - Temporary Structures

Temporary structures comprising the temporary bent W5LA on the Transition Structures, temporary bent E19AR on the Oakland Approach Structures, the Detours and Viaduct Modification (Initial) shall be designed for the response spectrum specified under 7.1.1.1 Functional Evaluation Earthquake (FEE).

### 7.2 PERFORMANCE CRITERIA

#### 7.2.1 Permanent Structures

The desired performance shall be defined in terms of two levels of earthquake:

- A functional evaluation earthquake (FEE)
- A safety evaluation earthquake (SEE)

#### 7.2.2 Functional Evaluation Earthquake (FEE)

After a functional evaluation earthquake the bridge may suffer "some to no loss of operation", with minor damage to structure.

SDCB

- Minor damage implies essentially elastic performance, and is characterized by:
- Minor inelastic response
- Narrow cracking in concrete
- No apparent permanent deformations
- Damage to expansion joints that can be temporarily bridged with steel plates

#### 7.2.3 Safety Evaluation Earthquake (SEE)

After a safety evaluation earthquake the damage to the bridge may be described as "minor to moderate damage with some loss of operation."

SDCB

Moderate damage implies visible and significant signs of damage with repairs or stabilization likely to be completed under emergency contracts and is characterized by:

- Minimal damage to the superstructure
- Limited damage to piers, including yielding of reinforcement and spalling of concrete cover
- Minimal damage to Piles and Pile caps
- Small permanent deformations, not interfering with serviceability of the bridge
- Damage to expansion joints that can be temporarily bridged with steel plates

#### 7.2.4 Limited Ductility Structure

The bridge shall be designed as a limited ductility structure as defined in ATC-32 with stable response to ground motions equivalent to the safety evaluation earthquake.

ATC-32, 3.21.3

**7.3.4 Soil-Structure Interaction Effects**

Soil-structure interaction effects shall be considered in all analyses. Foundation dynamic characteristics shall be incorporated into the analysis with discrete elements representing piles and footings with appropriate representation of the effects of soil structure interaction (SSI).

SDCB

**7.4 COMBINATION OF EFFECTS**

For response spectrum analysis, seismic effects from excitation in three orthogonal directions shall be combined by the 30% rule, where the forces resulting from excitation in one direction are combined with 30% of the forces resulting from excitation in the orthogonal directions.

ATC-32 3.21.9.2

**7.5 PERMANENT DISPLACEMENTS**

Maximum permanent displacement demands following the safety evaluation earthquake shall be obtained directly from non-linear time history analyses for the six 3-dimensional input motions.

ACT-32 3.21.10.3

The estimated permanent displacement of the bridge, as established by the nonlinear dynamic analysis, shall not exceed 300mm (12 inches) at the deck level.

To ensure the structural integrity and serviceability of the bridge after the safety evaluation earthquake, permanent vertical settlements at the pile caps shall not exceed 50mm (2 inches).

**7.6 CAPACITY DESIGN**

Structural elements, other than columns constituting the ductile lateral load resisting part of the structure, shall be designed to resist the over-strength capacity of the ductile elements.

ATC-32 3.21.14, 10.19.3  
 10.20, 10.24.8

**7.7 P-Δ EFFECTS**

P-Δ effects in Piers maybe ignored when:

$$W\delta_u \leq 0.25V_0H$$

where:

W= weight on the pier

$\delta_u$ = maximum drift at the top of the pier based on elastic response spectrum analysis

$V_0$ = base shear strength of the pier

H= height of the pier

**7.8 STRENGTH REDUCTION FACTORS**

Strength reduction factors shall be per ATC-32.

ATC-32 8.16.1.2.2

**7.9 MATERIAL PROPERTIES FOR DUCTILE ELEMENTS**

**7.9.1 Design flexural strength**

The design flexural strength of plastic hinges shall be based on expected material strengths:

$$f_{ce}=1.3f'_c$$

$$f_{ye}=1.1f_y$$

ACT-32 8.16.2.4

Maximum concrete strains at the design flexural strength shall not exceed 0.004. If moment curvature analysis is used to determine the design flexural strength, the steel strains shall be limited to 0.015.

### 7.9.2 Maximum plastic moment (Overstrength)

The maximum plastic (or overstrength) moment shall be calculated using one of the following two methods:

1. The maximum plastic (overstrength) moment shall be based on maximum feasible material strengths:

$$f_{co}=1.7f'_c$$
$$f_{yo}=1.25f_y$$

Maximum plastic (overstrength) moments shall be determined from moment-curvature analysis that considers the effects of concrete confinement and strain hardening of the reinforcement. The maximum plastic (overstrength) moment shall be the calculated moment at the design deformation of the element.

2. The maximum plastic (overstrength) moment of sections shall be based on expected material strengths:

$$f_{ce}=1.3f'_c$$
$$f_{ye}=1.1f_y$$

Maximum plastic (overstrength) moments shall be determined from moment-curvature analysis that considers the effects of concrete confinement and strain hardening of the reinforcement. The maximum plastic (overstrength) moment shall be 1.15 times the calculated moment at the design deformation of the element.

### 7.10 DEFORMATION CAPACITY

The deformation capacity of structures shall be calculated using plastic hinge lengths calculated according to ATC-32 8.18.2.4.2 and rotational capacities corresponding to the allowable material strains from Section 7.11.

For the piers, the allowable material strains shall be assumed to be the average extreme fiber strains over the plastic hinge length. For the piles, the allowable strains shall be assumed to be the maximum extreme fiber strains anywhere within the plastic hinge.

### 7.11 ALLOWABLE STRAINS

#### 7.11.1 Normal weight concrete

Allowable strains in normal weight concrete shall be:

Piers (Average extreme fiber strains in plastic hinge):

Functional evaluation earthquake  $\epsilon_c^{\text{Func}}=0.004$

Safety evaluation earthquake  $\epsilon_c^{\text{Safety}}=2/3\epsilon_{cu}$

where  $\epsilon_{cu}$  is the ultimate concrete strain according to the Mander model.

ATC-32, 8.16.4.4

(Mander et. al. J. Struct.  
Engineering, ASCE, 1988  
114(8), 0 1804-1849)

Piles (maximum extreme fiber strains in potential plastic hinge):  
 Functional evaluation earthquake  $\epsilon_c \text{Func} = 0.004$   
 Safety evaluation earthquake  $\epsilon_c \text{Safety} = 0.01$

**7.11.2 Reinforcing Steel**

Allowable strains in reinforcing steel shall be:

Piers (Average extreme fiber strains in plastic hinge):  
 Functional evaluation earthquake  $\epsilon_s \text{Func} = 0.015$   
 Safety evaluation earthquake  $\epsilon_s \text{Safety} = \frac{2}{3} \epsilon_{su}$   
 Where  $\epsilon_{su}$  is the steel strain at ultimate stress. For Grade 60 (A706) reinforcement,  $\epsilon_{su}$  may be taken as:  
 Confinement bars No. 10-32 (No. 3-10)  $\epsilon_{su} = 0.12$   
 Main bars No. 29-57 (No. 9-18)  $\epsilon_{su} = 0.09$

Piles (Maximum extreme fiber strains in potential plastic hinge):  
 Functional evaluation earthquake  $\epsilon_s \text{Func} = 0.015$   
 Safety evaluation earthquake  $\epsilon_s \text{Safety} = 0.02$   
 Where  $\epsilon_{su}$  is the steel strain at ultimate stress. For Grade 60 (A706) reinforcement,  $\epsilon_{su}$  may be taken as:  
 Confinement bars No. 10-32 (No. 3-10)  $\epsilon_{su} = 0.12$   
 Main bars No. 29-57 (No. 9-18)  $\epsilon_{su} = 0.09$

Grade 60 hardening strains may be taken as :  
 Bars No. 10-25 (No. 3-8)  $\epsilon_{sh} = 0.0150$   
 Bars No. 29-36 (No. 9-11)  $\epsilon_{sh} = 0.0100$   
 Bars No. 43 (No. 14)  $\epsilon_{sh} = 0.0075$   
 Bars No. 57 (No. 18)  $\epsilon_{sh} = 0.0050$

**7.12 DESIGN FOR JOINT SHEAR**

Superstructure/pier joints, pier/footing joints, and other joints between flexural members shall be proportioned and reinforced for joint shear in accordance with section 8.34 of ATC-32, 7.4 and 7.7 of SDC

**7.13 COLUMN SHEAR DESIGN**

Shear demands shall be the maximum plastic (or overstrength) shear obtained using the maximum plastic (overstrength) moment capacity of the column. Shear capacity shall be determined according to Section 3.6 of the Caltrans Seismic Design Criteria.

**8. GEOTECHNICAL AND FOUNDATION DESIGN**

The design of the foundations will be based on the Foundation Reports from Fugro and Earth Mechanics.

ATC-32 C3.21.11

ATC 32, SDC

SDC

Foundation Report

