San Francisco-Oakland Bay Bridge
East Span Seismic Safety Project

Contract 59A0040

DESIGN CRITERIA

Skyway Structures
March 1, 2001

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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	"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", 16th Edition, 1992	"AASHTO"
• "AASHTO LRFD Bridge Design Specifications" 2nd Edition, 1998.	"AASHTO LRFD"
• "Sacramento Light Rail Transit 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"
• "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 20th Ed. 1993	
• Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design – API RP2A-LRFD 1st Ed. 1993	
• AISC Manual of Steel Construction, 9th Edition	
• ANSI/ASCE 7-95 Standard	
• Stability Design Criteria – 5th Edition, SSRC	
• "SFOBB East Spans; Seismic Design Criteria Basis,"(SDCB) June 27, 2000	
• Technical Specifications for Skyway Structures	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 kg/m^3 (155 lb/ft^3).
The in-service air dry unit mass of sand lightweight concrete including reinforcement shall be 1920 kg/m^3 (120 lb/ft^3).

2.1.2 Steel

The unit mass of structural steel, including fabricated plate steel, rolled shapes, and wire, shall be 7850 kg/m^3 (490 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^2 (25 lb/ft^2)
Bike path– 13 mm	0.31 kN/m^2 (6 lb/ft^2)
Skyway Concrete Box Girder – 20 mm	0.47 kN/m^2 (10 lb/ft^2)

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 β_{LE} shall represent estimated peak hour traffic predictions for the year 2025.

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

AASHTO Seg. 6.4.3

Normal-weight Concrete	10.8×10^{-6} /°C	(6.0×10^{-6} /°F)
Sand Light-weight Concrete	9.0×10^{-6} /°C	(5.0×10^{-6} /°F)
Steel	11.7×10^{-6} /°C	(6.5×10^{-6} /°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 Segmental Concrete Bridges, shall correspond to the values listed below.

Positive Gradient		Negative Gradient		
T_1 =	30.0° C (54° F)	T_1 =	-15.0° C (-27° F)	
T_2 =	7.8° C (14° F)	T_2 =	-3.9° C	(-7° F)
T_3 =	0° C (0° F)	T_3 =	0° C	(0° F)
A =	300 mm (12 in)	A =	300 mm (12 in)	

2.4.4 Steel Temperature gradient - DT_s

For steel structures, the gradient shall be as shown in Figure 3.12.3.2 of the AASHTO LRFD Specifications for Highway Bridge Design.

AASHTO LRFD 3.12.3

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 project Hydraulics Report.

2.6 WIND – W

BDS 3.15

The Skyway shall be designed for wind loads as per BDS and ANSI/ASCE 7-95.

BDS 3.15
ANSI/ASCE 7-95

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.

In this event, the nominal stresses and/or loads and resistance factors of BDS shall be met by all bridge components.

2.7 SHIP COLLISION – SC

AASHTO Vessel Collision Design of Highway Bridges

"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.

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:

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

where

N	Group No.
γ	Load Factor, see Table 2.8.1A and 2.8.1B
β	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
α_{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

AASHTO Seg 7.2.2

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

AASHTO Seg 7.4 Table 7-2

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.

2.8.4 Construction Load Combinations, Load Factor Design Check – Concrete

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

AASHTO Seg 7.4.1 7.4.3

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

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 = γ (1.0 D + 1.0 P + 1.0 B + 1.0 SF + 1.0 E)

where

γ	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:

Group VIIA = γ (1.0 D + β_{LE} $(L + I)_H$ + 1.0 E + 1.0 B + 1.0 SF + 1.0 PS + 1.0 EQ)

Group XIA = γ (1.0 D + 1.0 E + 1.0 B + 1.0 SF + 1.0 PS + 1.0 P)

where

γ	1.0 for all design methods
β_{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	γ factor	D	L+I	CF	E	B	SF	W	WL	LF	PS	R+S	T	DT	α_{LRT} $L_{LRT}+I_{LRT}$	α_{LRT} CF_{LRT}	α_{LRT} NF_{LRT}	α_{LRT} LF_{LRT}	α_{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_{RT}	1.0	1	0	0	1	1	1	0	0	0	1	1	0	0	1	1	1	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 7.2.2

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
α_{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
LF_{LRT}	Longitudinal Force, LRT Train
NF_{LRT}	Nosing Force, LRT Train
WL_{LRT}	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	γ factor	D	(L+I)H	(L+I)P	CF	E	B	SF	W	WL	LF	PS	R+S	T	DT	EQ	α_{LRT} $L_{LRT}+I_{LRT}$	α_{LRT} CF_{LRT}	α_{LRT} NF_{LRT}	α_{LRT} LF_{LRT}	α_{LRT} WL_{LRT}	RT_2 or BR	LR_2 or BR	α_{LRT} DR_{LRT}+ I
I	1.30	β_D	1.67	0	1	β_E	1	1	0	0	0	0.77	0	0	0	0	1	1	0	0	0	0	0	0
I_{P3D}	1.30	β_D	1	1.25	1	β_E	1	1	0	0	0	0.77	0	0	0	0	1	1	1	0	0	0	0	0
II	1.30	β_D	0	0	0	β_E	1	1	1	0	0	0.77	0	0	0	0	0	0	0	0	0	0	0	0
III	1.30	β_D	1	0	1	β_E	1	1	β_{w1}	1	1	0.77	0	0	0	0	1	1	1	1	1	0	1	0
IV	1.30	β_D	1	0	1	β_E	1	1	0	0	0	0.77	1	1	0.5	0	1	1	0	0	0	1	1	0
V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	0.80	1	1	1	0	1	0	0	0	0	1	1	0
VI	1.25	β_D	1	0	1	β_E	1	1	β_{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	β_E	1	1	0	0	0	1	0	0	0	1	0	0	0	0	0	0	0	0
XI	1.30	β_D	1	0	1	β_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

β_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.
β_D =1.00 when checking columns for maximum axial load and associated moment.
β_E = 0.5 for checking positive moments in rigid frames.
β_E = 1.00 for vertical earth pressure.
β_E = 1.30 for lateral earth pressure
β_{w1} = 0.3 for highway/bicycle/pedestrian loads only
β_{w1} = 1.3 for LRT+highway/bicycle/pedestrian loads only

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
α_{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
BR	Broken rail
RT_2	Radial thermal rail force
LT_2	Long. Restraint forces
DR	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, $\gamma \times \beta_d$ 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

AASHTO Seg. 9.1

Tensile stresses in prestressing tendons shall not exceed the following:

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	$0.83\, f^*_y$
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

AASHTO Seg 9.2.1.1

Normal weight and Sand lightweight concrete — $0.6\, f'_{ci}$

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_{sy}; internal tendons:

AASHTO Seg 9.2.1.2

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

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_{sy}; 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)	AASHTO Seg Table 7-2
	[$6\sqrt{f'_{ci}}$ (psi)]	
Sand lightweight concrete	$0.42\sqrt{f'_{ci}}$ (MPa)	
	[$0.85 \times 6\sqrt{f'_{ci}}$ (psi)]	

3.1.2.2 Stresses at service level after losses - fully prestressed components

3.1.2.2.1 Compression

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

3.1.2.2.2 Tension in the precompressed tensile zone

Longitudinal and transverse stresses, *with* 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)	AASHTO Seg 9.2.2.2
	[$3\sqrt{f'_c}$ psi)]	
Sand lightweight concrete	$0.21\sqrt{f'_c}$ (MPa)	
	[$0.85 \times 3\sqrt{f'_c}$ (psi)]	

Longitudinal and transverse stresses, *without* 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 *with* or *without* minimum bonded auxiliary reinforcement through the joints:

Normal weight and Sand lightweight concrete	0 tension allowed	BDS 9.15.2.2

3.1.2.2.3 Tension in other areas

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)	AASHTO Seg. 9.2.2.4
	[$6\sqrt{f'_c}$ (psi)]	
Sand lightweight concrete	$0.42\sqrt{f'_c}$ (MPa)	
	[$0.85 \times 6\sqrt{f'_c}$ (psi)]	

3.1.2.3 Cracking stress

Modulus of rupture:		BDS 9.15.2.3
Normal weight concrete	$0.63\sqrt{f'_c}$ (MPa)	
	[$7.5\sqrt{f'_c}$ (psi)]	

Sand lightweight concrete	$0.52\sqrt{f'_c}$ (MPa) [$0.85 \times 7.5\sqrt{f'_c}$ (psi)]	

3.1.2.4 Bearing stress

Post-tensioned anchorage at service level	21 MPa (3000 psi)	BDS 9.15.2.4

3.1.2.5 Shear

Shear design shall follow the AASHTO LRFD criteria based on modified compression field theory. — AASHTO LRFD 1993

3.1.2.6 Web Shear

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)]

3.2 STRENGTH REDUCTION FACTORS

Superstructure Strength Reduction Factors, ϕ, 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 $\varepsilon_{ult} = 224 \times 10^{-6}$ 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^*_y = 1674 MPa	(243 ksi)
Modulus of Elasticity	E= 197,000 MPa	(28,500 ksi)
Anchor set	Δ = 6 mm	(1/4 inch)

3.3.2.1.2 Prestress losses

Standard galvanized steel ducts:

Wobble coefficient	κ = 0.0007 /m	(0.0002 /ft)
Friction coefficient	μ = 0.2 /rad	

Deviated polyethylene duct (external to concrete):

Wobble coefficient	κ = 0.0007 /m	(0.0002 /ft)
Friction coefficient	μ = 0.25 /rad	

Prestressing steel may be bonded, internal to the section, or unbonded, external to the section.

3.3.2.2 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	Δ = 2 mm	(1/16 inch)
Friction coefficient	μ = 0.30 /rad	
Wobble coefficient	κ = 0.0007 /m	(0.0002 /ft)

3.3.3 Mild Steel Reinforcement

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)

4. CONCRETE – CAST IN PLACE SUPERSTRUCTURE & 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 Principle 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 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)

4.3.1.3 Piers

28-day minimum strength	f'_c = 30 MPa	(4350 psi)

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	Δ = 6 mm	(1/4 inch)

4.3.2.1.2 Prestress losses

For conventional CIP Concrete on Falsework:

Standard galvanized steel ducts:

Wobble coefficient	κ = 0.0007 (/m)	(0.0002 /ft)
Friction coefficient	μ = 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	Δ = 2 mm	(1/16 inch)
Friction coefficient	μ = 0.30 /rad	
Wobble coefficient	κ = 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
  $0 \le r \le 75mm$
  where r is the unsupported length of a strip seal supported by adjacent beams
- Functional Evaluation Earthquake
  $0 \le r \le 130mm$
- Safety Evaluation Earthquake
  Closing: $0 \le r$
  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

ATC-32 3.21.10.1, ATC-32 3.21.11

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 1½ may be used for piles.

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

SDCB

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.

7.4 COMBINATION OF EFFECTS

ATC-32 3.21.9.2

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.

7.5 PERMANENT DISPLACEMENTS

ATC 32 3.21.10.3

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)

7.6 CAPACITY DESIGN

ATC-32 3.21.14, 10.19.3, 10.20, 10.24.8

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.

7.7 HINGE BEAMS

ATC-32 3.21.12, 10.29.8

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.

7.8 P-D EFFECTS

ATC-32 3.21.15

P-Δ effects in Piers maybe ignored when:

$Wd_u \leq 0.25 V_0 H$

where:

W = weight on the pier
δ_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

7.9 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:

ATC-32 8.16.2.4.1

$f'_{ce} = \mathbf{1.3}\, f_c$
$f_{ye} = \mathbf{1.1}\, f_y$

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:

   $f'_{co} = \mathbf{1.7}\, f'_c$
   $f_{yo} = \mathbf{1.25}\, f'_y$

   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:

   $f'_{co} = \mathbf{1.3}\, f'_c$
   $f_{yo} = \mathbf{1.1}\, f'_y$

   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 stains 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 $\varepsilon_c^{Func} = 0.004$
Safety evaluation earthquake $\varepsilon_c^{Safety} = 2/3\ \varepsilon_{cu}$
where ε_{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 $\varepsilon_c^{Func} = 0.004$
Safety evaluation earthquake $\varepsilon_c^{Safety} = 0.01$

7.12.2 Reinforcing Steel

ATC-32 C3.21.11.1

Allowable strains in reinforcing steel shall be:

Piers (Average extreme fiber strains in plastic hinge):
Functional evaluation earthquake $\varepsilon_s^{Func} = 0.015$
Safety evaluation earthquake $\varepsilon_s^{Safety} = ½\ \varepsilon_{su}$
Where ε_{su} is the steel strain at ultimate stress. For Grade 60 (A706) reinforcement, ε_{su} may be taken as:
Confinement bars No. 10 – 25 (No. 3 – 8) ε_{su}=0.12
Main bars No. 29 – 57 (No. 9 – 18) ε_{su}=0.09

Piles (Maximum extreme fiber strains in potential plastic hinge):
Functional evaluation earthquake $\varepsilon_s^{Func} = 0.015$
Safety evaluation earthquake $\varepsilon_s^{Safety} = 0.02$
Where ε_{su} is the steel strain at ultimate stress. For Grade 60 (A706) reinforcement, ε_{su} may be taken as:
Confinement bars No. 10 – 25 (No. 3 – 8) $\varepsilon_{su} = 0.12$
Main bars No. 29 – 57 (No. 9 – 18) $\varepsilon_{su} = 0.09$

Grade 50 hardening strains may be taken as:
Bars No. 10 – 25 (No. 3 – 8) $\varepsilon_{sh} = 0.0150$

Bars No. 29 – 36 (No. 9 – 11)	$\varepsilon_{sh} = 0.0100$
Bar No. 43 (No. 14)	$\varepsilon_{sh} = 0.0075$
Bar No. 57 (No. 18)	$\varepsilon_{sh} = 0.0050$

7.12.3 Structural Steel Pile Shells (Casings)

Functional evaluation earthquake	$\varepsilon_s^{Func} = 0.015$
Safety evaluation earthquake	$\varepsilon_s^{Safety} = 0.02$

7.13 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.

ATC 32 8.34

7.14 COLUMN SHEAR DESIGN

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.

8. GEOTECHNICAL AND FOUNDATION DESIGN

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