LAO PEOPLE'S DEMOCRATIC REPUBLIC
PEACE INDEPENDENCE DEMOCRACY UNITY PROSPERITY

MINISTRY OF PUBLIC WORKS AND TRANSPORT

DEPARTMENT OF ROADS

Lao Road Sector Project2 (LRSP2)

Consulting Services for Conceptual Engineering Design of Improvement & Maintenance of NR 13 South from KM 71 to KM 346

KM 71+300 –KM 346+000

VOLUME 5

CONCEPTUAL DESIGN REPORT

ANNEX A

BRIDGE DESIGN REPORT

(FINAL)

CONSULTANT:

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DESIGN CRITERIA

1. Design Phylosophy

(1) General

Bridges and structures shall be designed for specified limit states to achieve the following objectives.

* Design life of structures shall be 100 years as specified in the Specifications for Bridge Design as following:
  1. Specification for Bridge Design based on AASHTO LRFD 2007
  2. Road Design Manual of Ministry of Public Works and Transport (Lao People's Democratic Republic) April 2018
* The bridges have attractive appearance, are cost-effective and incorporate modern construction methods.
* The construction maximizes the use of local materials and Lao construction resources.

(2) Limit States

Each component and connection shall satisfy the following equation for service limit, fatigue and fracture limit, strength limit and extreme event limit states.

$$Q = \Sigma \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

where:

γ_i = load factor

ϕ = resistance factor

η_i = load modifier

= $\eta_D * \eta_R * \eta_I$

where:

η_D = a factor relating to ductility

η_R = a factor relating to redundancy

η_I = a factor relating to operational importance

These factors are all taken as 1.0 as the project bridges are conventional design and level, and typical bridges.

Factor	Category	Strength Limit State	Other Limit States
η_D	For nonductile components and connections	$\geq$ 1.05	1.00
	For conventional designs and details complying with AASHTO	1.00	
	For components and connections for which additional ductile-enhancing measures have been specified beyond those required by AASHTO	$\geq$ 0.95	
η_R	For nonredundant members	$\geq$ 1.05	1.00
	For conventional levels of redundancy	1.00	
	For exceptional levels of redundancy	$\geq$ 0.95	
η_I	For important bridges	$\geq$ 1.05	1.00
	For typical bridges	1.00	
	For relatively less important bridges	$\geq$ 0.95	

Q_i = force effect

Q = factored load

R_n = nominal resistance

R_r = factored resistance

All limit states shall be considered of equal importance.

2. Load Factor and Load Combination, Resistance Factors

The total factored force effect shall be taken as:

$$Q = \Sigma\, \eta_i * \gamma_i * Q_i$$

Q_i = force effects from loads

γ_i = load factors specified in Tables 2-1 and 2-2

Table 2-1. Load Combinations and Load Factors

Load Combination / Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These At a Time		
										EQ	CT	CV
STRENGTH-I	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-
STRENGTH-II	γ_p	1.35				1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-
STRENGT	γ_p	-		1.4		1.0	0.50/1	γ_{TG}	γ_{SE}	-	-	-

H-III				0		0	.20					
STRENGTH-IV	γ_p 1.50	-				1.00	0.50/1.20	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-
EXTREME-I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-
EXTREME-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-
SERVICE-IV	1.00	-	1.00	0.70	-	1.00	1.00/1.20	-	1.00	-	-	-
FATIGUE	-	0.75	-	-	-	-	-	-	-	-	-	-

Table 2-2. Load Factors for Permanent Loads, γ_p

Type of Load			Load Factor	
			Maximum	Minimum
DC	:	Component and Attachments	1.25	0.90
DD	:	Downdrag	1.40	0.25
DW	:	Wearing Surfaces and Utilities	1.50	0.65
EH	:	Horizontal Earth Pressure		
	*	Active	1.50	0.90
	*	At-Rest	1.35	0.90
EL	:	Locked-in Erection Stress	1.00	1.00
EV	:	Vertical Earth Pressure		
	*	Overall Stability	1.00	N/A
	*	Retaining Walls and Abutments	1.35	1.00
	*	Rigid Buried Structure	1.30	0.90
	*	Rigid Frames	1.35	0.90
	*	Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
	*	Flexible Metal Box Culverts	1.50	0.90
ES	:	Earth Surcharge	1.50	0.75

Table 2-3. Load Factor for Temperature Gradient γ_{TG}

γ_{TG}	Conditions
0.00	: at the strength and extreme event limit states
1.00	: at the service limit state when live load is not considered
0.50	: at the service limit state when live load is considered

Among these load combinations above, the following nine (9) cases shall be considered for project bridges except for Owner-specified special case (STRENGTH-II), case for steel structure (SERVICE-II) and case for prestressed substructure (SERVICE-IV).

Table 2-4 Load Cases to be Considered in the Project

Limit State	Outline of Limit State	**Adopt**
STRENGTH – I	: Basic load combination relating to the normal vehicular use of the bridge without wind.	○
STRENGTH – II	Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.	
STRENGTH – III	: Load combination relating to the bridge exposed to wind velocity exceeding 90 km/h without live load.	○
STRENGTH – IV	: Load combination relating to very high dead load to live load force effect ratio. (for bridges with spans exceeding 60 m)	○
STRENGTH – V	: Load combination relating to normal vehicular use of the bridge with wind of 90 km/h velocity.	○
EXTREME EVENT – I	: Load combination including earhquake.	○
EXTREME EVENT – II	: Load combination relating to collision by vessels and vehicles with a reduced live load other than that which is part of the vehicular collision load, CT.	○
SERVICE – I	: Load combination relating to the normal use of the bridge with a 90 km/h wind and all loads taken at their normal values, to control crack with in RC structure and to investigate compression in prestressed concrete components.	○
SERVICE – II	Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicles live load	
SERVICE – III	: Load combination relating only to tension in prestressed concrete superstructure with the objective of crack control.	○
SERVICE – IV	Load combination relating only to tension in prestressed concrete substructures with the objective of crack control	
FATIGUE	: Fatigue and fracture load combination relating to repetitive gravitational vehicular live load and dynamic responses under a single design truck.	○

3. Design Loads

The following permanent and transient loads shall be considered for bridges and road related structures:

Permanent Loads

DD = Downdrag

DC = Dead load of structural components and nonstructural attachemnt

DW = Dead load of wearing surfaces and utilities

EH = Horizontal earth pressure load

EL = Accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning

ES = Earth surcharge load

EV = Vertical pressure from dead load of earth fill

Transient Loads

BR	=	Vehicular braking force	LS	=	Live load surcharge
CE	=	Vehicular centrifugal force	PL	=	Pedestrian live load
CR	=	Creep	SE	=	Settlement
CT	=	Vehicular collision force	SH	=	Shrinkage
CV	=	Vessel collision force	TG	=	Temperature gradient
EQ	=	Earthquake	TU	=	Uniform temperature
FR	=	Friction	WA	=	Water load and stream pressure
IM	=	Vehicular dynamic load allowance	WL	=	Wind on live load
LL	=	Vehicular live load	WS	=	Wind load on structure

Among them, the main loads are as follows:

3.1 Dead Load: DC, DW and EV

Dead loads shall include the weight of all components of the structure, appurtenances and utilities attached thereto, earth cover, wearing surface and future overlays.

The following densities specified in Table 3-1 for each material is used for dead loads.

Table 3-1 Densities

Matrial		Density (kg/m3)
Alumilum Alloys		2800
Bituminous Wearing Surface		2250
Cast Iron		7200
Cinder Filling		960
Compacted Sand, Silt or Clay		1925
Concrete	Low-density	1775
	Sand-low-density	1925
	Normal Density with fc ’< 35 Mpa	2320
	Normal Density with 35 < fc ’< 105 Mpa	2240+2,29f’c
Loose Sand, Silt and Gravel		1600
Soft Clay		1600
Rolled Gravel, Macadam, or Ballast		2250
Steel		7850
Stone Masonry		2725
Wood	Hard	960
	Soft	800
Water	Fresh	100
	Salt	1025
Item		Mass per Unit length
Transit Rails, Ties, and Fastening per Track		0.3

For utilities, the following values in Table 3-2 are used from the previous practices in this Project.

Table 3-2 Unit Weight for Utilities

Utility	Unit	Weight	Remarks
Water Pipe	kg/m	To be investigated	To be studied for each bridge
Electric Line	kg/m	To be investigated	To be studied for each bridge
Bridge Railing	kg/m	50	

3.2 Live Loads

3.2.1 Gravity Loads: LL and PL

(1) Number of Design Lanes

The number of design lanes should be determined by taking the integer part of the ratio w/3600, where w is the clear roadway width in mm between curbs and/or barriers. The number of lanes are summarised in Table 3-3 for estimated project bridge roadway width.

Table 3-3 Number of Lanes and Lane Width

Clear Roadway Width (mm)	Number of Design Lanes	Traffic Lane Width (mm)
$w < 3600$	Actual Lane Numbers	Actual Lane Width
$3600 \le w < 6000$	1	Actual Lane Width
$6000 \le w < 7200$	2	w / 2
$7200 \le w < 10500$	2	3600
$10500 \le w < 14000$	3	3600

(2) Multiple Presence Factor

Trucks will be present in adjacent lanes on roadways with multiple design lanes but it is unlikely that three adjacent lanes will be loaded simultaneously with the heavy loads. Therefore some adjustments in design loads are necessary. To account for such effects, the following multiple presence factors “m” are provided as shown in Table 3-4.

Table 3-4 Multiple Presence factors “m”

Number of Loaded Lanes	1	2	3	> 3
Multiple Presence Factors “m”	1.20	1.00	0.85	0.65

However, these factors shall not be applied or shall be removed for the following cases.

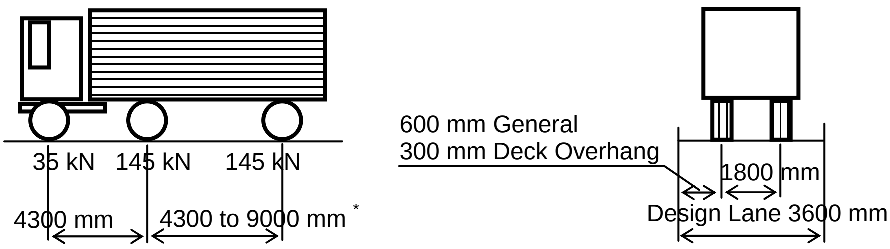
Table 3-5 Cases “m” shall not be applied or Removed

Cases “m” shall not be applied	Cases “m” shall be removed
1) Approximate load distribution factors, which are in Tables 3-8 to 3-14 except for Lever Rule, are used.	Approximate load distribution factors are used for fatigue limit state. (The factor 1.20 shall be removed from the approximate equations.)
2) Special requirements for exterior beams in beam-slab bridges are used.	
3) Pedestrian loads	

For the purpose of determining the number of lanes when the load combination includes the pedestrian loads combined with one or more lanes of vehicular live load, the pedestrian loads may be taken to be one loaded lanes.

(3) Design Vehicular Live Load

Vehicular live loading on the roadways of bridges, designated HL-93, shall consist of a combination of the followings:



* For fatigue load, the distance between 145 kN axles shall be constant of 9000 mm

Figure 3-1 Design Truck

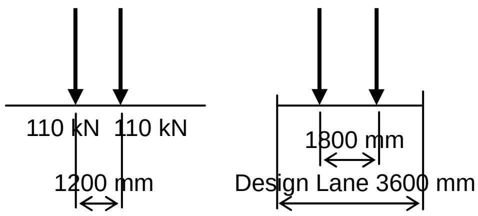


Figure 3-2 Design Tandem

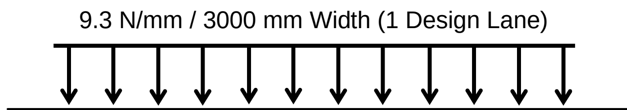


Figure 3-3 Design Lane Load

Combination -1	The effects of Design Tandem and Design Lane Load	Other than those in Combination -3
Combination -2	The effects of Design Truck with variable axle spacing and Design Lane Load	Other than those in Combination -3
Combination -3	90 % of Two Design Trucks spaced a minimum 15,000 mm between the lead axle of one truck and the rear axle of the other truck, combined with 90 % of Design Lane Load. The distance between the 145,000 N axles of each truck shall be 4,300 mm.	Negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only

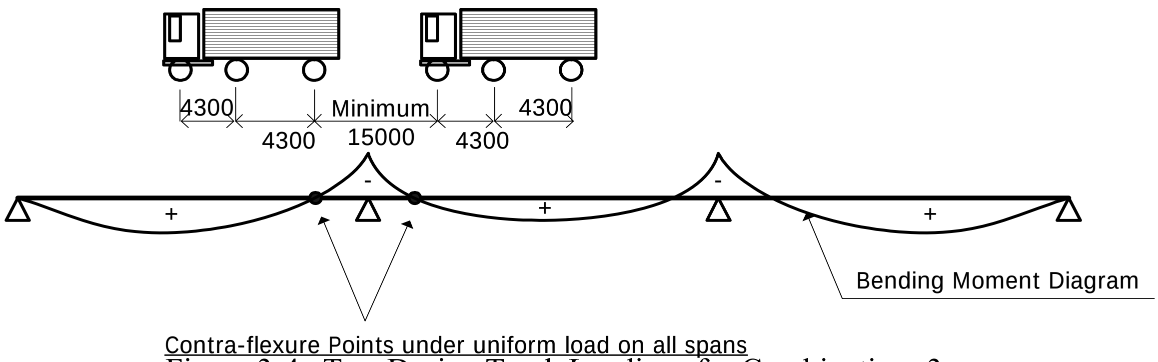


Figure 3-4 Two Design Truck Loadings for Combination -3

Moreover, the followings shall be considered in application of vehicular live loads.

1) Axles that do not contribute to the extreme force effect under consideration shall be negrected.

2) Both the design lanes and the 3000 mm loaded width in each lane shall be positioned to produce extreme force effects.

(4) Pedestrian Loads

For all sidewalks wider than 600 mm, <u>$3.6 * 10^{-3}$ MPa</u> of pedestrian loads shall be applied simultaneously with the vehicular design live loads.

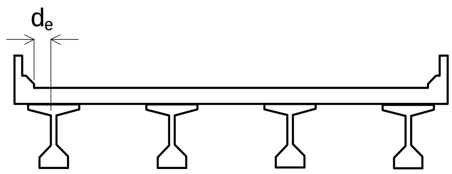
Bridges for only pedestrian and/or bicycle traffic shall be designed for a live load of $4.1*10^{-3}$ MPa

(5) Load Distribution

For beam-slab bridges as shown in Table 3-6 that meet the following conditions, load distribution factors specified in Tables 3-8 to 3-14 are used.

* Width of deck and cross-section are both constant;
* Number of beams is not less than four (4) ;
* Beams are parallel and have approximately the same stiffness;
* Roadway part of overhang, d_e, does not exceed 910 mm ;

  d_e = distance from exterior web of exterior beam and the interior edge of curb or traffic barrier



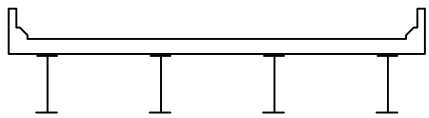
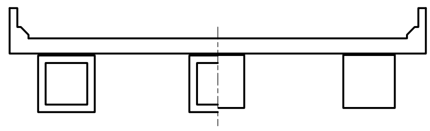
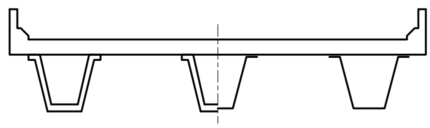
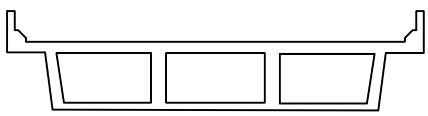
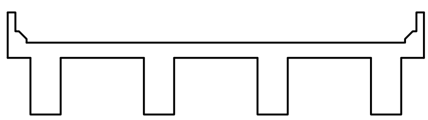
* Curvature in plan is less than the limit in terms of central angle subtended by one span;

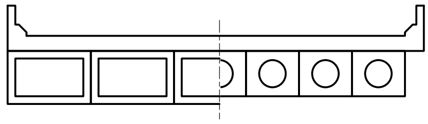
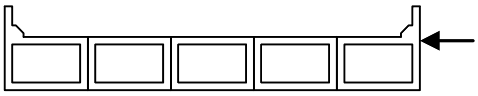
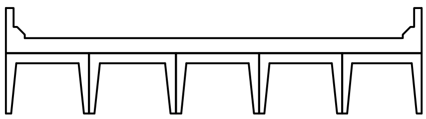
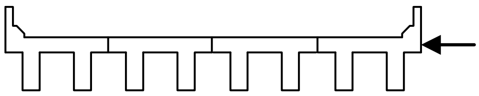
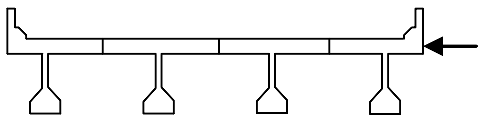
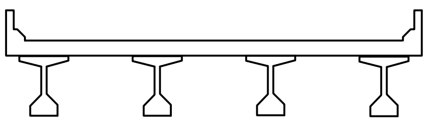
For torsionally stiff closed sections: $\leq 12^{o}$
For cast-in-place multicell concrete box girders: $\leq 34^{o}$
For open cross-sections:

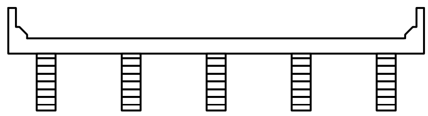
Number of Beams	One Span	Two or More Spans
2	2^{o}	3^{o}
3 or 4	3^{o}	4^{o}
5 or more	4^{o}	5^{o}

Where bridges meet these conditions, permanent loads of and on the deck can be considered distribute uniformly among the beams and/or stringers.

Table 3-6 Commom Deck Superstructures (1)

Supporting Components	Type of Deck	Typical Cross-Section
Steel Beam	Cast-in-place concrete slab, precast concrete slab, steel grid, glued/spiked panels, stressed wood	 (a)
Closed Steel or Precast Concrete Boxes	Cast-in-place concrete slab	 (b)
Open Steel or Precast Concrete Boxes	Cast-in-place concrete slab, Precast concrete dexk slab	 (c)
Cast-in-place Concrete Multicell Box	Monolithic concrete	 (d)
Cast-in-place Concrete Tee Beam	Monolithic concrete	

		(e)
Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys	Cast-in-place concrete overlay	 (f)
Precast Solid, Voided, or Cellular Concrete Box with Shear Keys and with/without Transverse Post-Tensioning	Integral concrete	 (g)
Precast Concrete Channel Sections with Shear Keys	Cast-in-place concrete overlay	 (h)
Precast Concrete Double Tee Section with Shear Keys and with/without Transverse Post-Tensioning	Integral concrete	 (i)
Precast Concrete Tee Section with Shear Keys and with/without Transverse Post-Tensioning	Integral concrete	 (j)
Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	 (k)

Wood Beams	Cast-in-place concrete or plank, glued/spiked panels or stressed wood	 (l)

For the following tables, span length "L" shall be taken as:

Table 3-7 L for Use in Live Load Distribution Factor Equations

Force Effect	L (mm)
① Positive Moment	L_1
② Negative Moment – Near interior supports of continuous spans from contraflexure points under a uniform load on all spans	$(L_1+L_2)/2$
③ Negative Moment – Other than interior supports of continuous spans	L_4
④ Shear	span under consideration
⑤ Exterior Reaction	L_1
⑥ Interior Reaction of Continuous Span	$(L_1+L_2)/2$

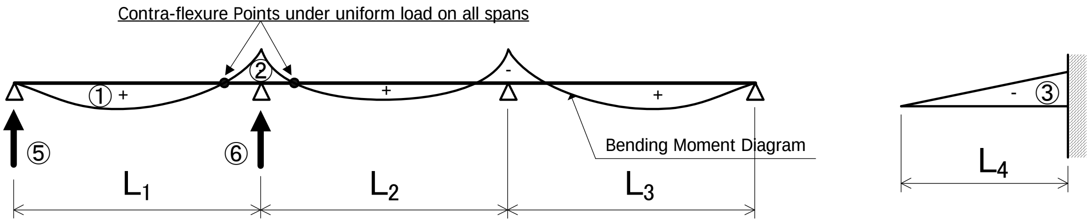


Table 3-8 Distribution of Live Loads Per Lane for Moment in Interior Beams with Concrete Deck (1)

Type of Beams	Applicable Cross-Section (Table 3.6)	Distribution Factors	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- & Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^{\ 3}}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^{\ 3}}\right)^{0.1}$	$1100 \le S \le 4900$ $110 \le t_s \le 300$ $6000 \le L \le 73000$ $Nb \ge 4$ $4*10^9 \le K_g \le 3 * 10^{12}$
		Use lesser of the values obtained from the equation above with Nb = 3 or the lever rule	$N_b = 3$
Cast-in-place Concrete Multicell Box	d	One Design Lane Loaded: $\left(1.75+\frac{S}{1100}\right)\left(\frac{300}{L}\right)^{0.35}\left(\frac{1}{N_c}\right)^{0.45}$	$2100 \le S \le 4000$ $18000 \le L \le 73000$ $N_c \ge 3$

		Two or More Design Lanes Loaded: $\left(\frac{13}{N_c}\right)^{0.3}\left(\frac{S}{430}\right)\left(\frac{1}{L}\right)^{0.25}$	If $N_c > 8$, use $N_c = 8$
Concrete Deck on Concrete Spread Box Beams	b, c	One Design Lane Loaded: $\left(\frac{S}{910}\right)^{0.35}\left(\frac{Sd}{L^2}\right)^{0.25}$ Two or More Design Lanes Loaded: $\left(\frac{S}{1900}\right)^{0.6}\left(\frac{Sd}{L^2}\right)^{0.125}$	$1800 \le S \le 5500$ $6000 \le L \le 43000$ $450 \le d \le 1700$ $N_b \ge 3$
		Use lever rule	S > 5500

Table 3-8 Distribution of Live Loads Per Lane for Moment in Interior Beams with Concrete Deck (2)

Type of Beams	Applicable Cross-Section (Table 3.6)	Distribution Factors	Range of Applicability
Concrete Beams used in Multibeam Decks	f	One Design Lane Loaded: $k\left(\frac{b}{2.8L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$ where: $k = 2.5\,(N_b)^{-0.2} \ge 1.5$ Two or More Design Lanes Loaded: $k\left(\frac{b}{7600}\right)^{0.6}\left(\frac{b}{L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}$	$900 \le b \le 1500$ $6000 \le L \le 37000$ $5 \le N_b \le 20$
	g if sufficiently connected to act as a unit		
	h	Regardless of Number of Loaded Lanes: S/D where: $C = K\,(W/L) \le K$ $D = 300\left[11.5 - N_L + 1.4N_L(1-0.2C)^2\right]$ when $C \le 5$ $D = 300\left[11.5 - N_L\right]$ when $C > 5$ $K = \sqrt{\frac{(1+\mu)I}{J}}$ for preliminary design, the following values of K may be used: Beam Type K	Skew $\le 45^{\circ}$ $N_L \le 6$
	g, i, j if connected only enough to prevent relative vertical displacement at the interface		

		Nonvoided rectangular beams 0.70 Rectangular beams with circular voids: 0.80 Box section beams 1.00 Channel beams 2.20 T-beam 2.00 Double T-beam 2.00	
Open Steel Grid Deck on Steel Beams	a	One Design Lane Loaded: S/2300 if $t_g < 100$ mm S/3050 if $t_g \geq$ 100 mm	$S \leq 1800$
		Two or More Design Lanes Loaded: S/2400 if $t_g < 100$ mm S/3050 if $t_g \geq$ 100 mm	$S \leq 3200$
Concrete Deck on Multiple Steel Box Girders	b, c	Regardless of Number of Loaded Lanes: $0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L}$	$0.5 <= \frac{N_L}{N_b} <= 1.5$

Table 3-9 Distribution of Live Loads Per Lane for Moment in Exterior Beams with Concrete Deck

Type of Superstructure	Applicable Cross-Section (Table 3.6)	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- & Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	Lever Rule	$g = e\, g_{interior}$ $e = 0.77 + \frac{d_e}{2800}$	$-300 \leq d_e \leq 1700$
			Use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

Cast-in-place Concrete Multicell Box	d	$g = \frac{W_e}{4300}$	$g = \frac{W_e}{4300}$	$W_e \le S$
		or the provisions for a whole-width design specified in Article 3.6.2.2.1		
Concrete Deck on Concrete Spread Box Beams	b, c	Lever Rule	$g = e\, g_{interior}$ $e = 0.97 + \frac{d_e}{8700}$	$0 \le d_e \le 1400$ $1800 < S \le 5500$
			Lever Rule	$S > 5500$
Concrete Box Beams used in Multibeam Decks	f, g	Lever Rule	$g = e\, g_{interior}$ $e = 1.04 + \frac{d_e}{7600}$ ≥ 1.0	$d_e \le 600$
Concrete Beams other than Box Beams used in Multibeam Decks	h i, j if connected only enough to prevent relative vertical displacement at the interface	Lever Rule	Lever Rule	N/A
Open Steel Grid Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A
Concrete Deck on Multiple Steel Box Girders	b, c	Same as for interior beams		

Table 3-10

Reduction of Load Distribution Factors for Moment in Longitudinal Beams on Skewed Support

Type of Superstructure	Applicable Cross-Section (Table 3.6)	Any Number of Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams;	a, e, k and also i, j if sufficiently connected to act as a unit	$1 - c_1(\tan\theta)^{1.5}$ $c_1 = 0.25\left(\frac{K_g}{L t_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$ If $\theta < 30^o$ then c1 = 0.0 If $\theta > 60^o$ use $\theta = 60^o$	$30^o \le \theta \le 60^o$ $1100 \le S \le 4900$ $6000 \le L \le 73000$ $N_b \ge 4$

Concrete T-Beams, T- & Double T-Sections			
Concrete Deck on Concrete Spread Box Beams, Cast-in-Place Multicell Box Concrete Box Beams and Double T- Sections used in Multibeam Decks	b, c, d, f, g	$1.05-0.25\tan\theta\leq1.0$ If $\theta > 60^o$ use $\theta = 60^o$	$0^o \leq \theta \leq 60^o$

Table 3-11 Distribution of Live Load per Lane for Transverse Beams for Moment and Shear

Type of Deck	Fraction of Wheel Load to Each Floorbeam	Range of Application
Plank	S/1200	N/A
Laminated Wood Deck	S/1500	$S \leq 1500$
Concrete	S/1800	$S \leq 1800$
Steel Grid & Unfilled Grid Deck Composite with Reinforced Concrete Slab	S/1400	$t_g \leq 100$ $S \leq 1500$
Steel Grid & Unfilled Grid Deck Composite with Reinforced Concrete Slab	S/1800	$t_g \geq 100$ $S \leq 1800$
Steel Bridge Corrugated Plank	S/1700	$t_g \geq 50$

Table 3-12 Distribution of Live Loads Per Lane for Shear in Interior Beams

Type of Superstructure	Applicable Cross-Section (Table 3.6)	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- & Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$0.36+\frac{S}{7600}$	$0.20+\frac{S}{3600}-\left(\frac{S}{10700}\right)^{2.0}$	$1100 \leq S \leq 4900$ $6000 \leq L \leq 73000$ $110 \leq t_s \leq 300$ $N_b \geq 4$
		Lever Rule	Lever Rule	$N_b = 3$
Cast-in-place Concrete Multicell Box	d	$\left(\frac{S}{2900}\right)^{0.6}\left(\frac{d}{L}\right)^{0.1}$	$\left(\frac{S}{2200}\right)^{0.9}\left(\frac{d}{L}\right)^{0.1}$	$1800 \leq S \leq 4000$ $6000 \leq L \leq 73000$

				$890 \le d \le 2800$ $N_c \ge 3$
Concrete Deck on Concrete Spread Box Beams	b, c	$\left(\frac{S}{3050}\right)^{0.6}\left(\frac{d}{L}\right)^{0.1}$	$\left(\frac{S}{2250}\right)^{0.8}\left(\frac{d}{L}\right)^{0.1}$	$1800 \le S \le 5500$ $6000 \le L \le 43000$ $450 \le d \le 1700$ $N_b \ge 3$
		Lever Rule	Lever Rule	$S > 5500$
Concrete Box Beams used in Multibeam Decks	f, g	$0.70\left(\frac{b}{L}\right)^{0.15}\left(\frac{I}{J}\right)^{0.05}$	$\left(\frac{b}{4000}\right)^{0.4}\left(\frac{b}{L}\right)^{0.1}\left(\frac{I}{J}\right)^{0.05}$	$900 \le b \le 1500$ $6000 \le L \le 37000$ $5 \le Nb \le 20$ $1.0 * 10^{10} \le J \le 2.5 * 10^{11}$ $1.7 * 10^{10} \le I \le 2.5 * 10^{11}$
Concrete Beams other than Box Beams used in Multibeam Decks	h i, j if connected only enough to prevent relative vertical displacement at the interface	Lever Rule	Lever Rule	N/A
Open Steel Grid Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A
Concrete Deck on Multiple Steel Box Girders	b, c	Same as for Moment		

Table 3-13 Distribution of Live Loads Per Lane for Shear in Exterior Beams

Type of Superstructure	Applicable Cross-Section (Table 3.6)	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid or Unfilled Grid Deck Composite with Reinforced Concrete Slab on	a, e, k and also i, j if sufficiently connected to act as a unit	Lever Rule	$g = e\ g_{interior}$ $e = 0.6 + \frac{d_e}{3000}$	$-300 \le d_e \le 1700$
			Lever Rule	$N_b = 3$

Steel or Concrete Beams; Concrete T-Beams, T- & Double T-Sections				
Cast-in-place Concrete Multicell Box	d	Lever Rule	$g = e\, g_{interior}$ $e = 0.64 + \frac{d_e}{3800}$	$-600 \le d_e \le 1500$
		or the provisions for a whole-width design specified in Article 3.6.2.2.1		
Concrete Deck on Concrete Spread Box Beams	b, c	Lever Rule	$g = e\, g_{interior}$ $e = 0.80 + \frac{d_e}{3050}$	$0 \le d_e \le 1400$
			Lever Rule	S > 5500
Concrete Box Beams used in Multibeam Decks	f, g	Lever Rule	$g = e\, g_{interior}$ $e = 1.02 + \frac{d_e}{15000}$ ≥ 1.0	$d_e \le 600$
Concrete Beams other than Box Beams used in Multibeam Decks	h i, j if connected only enough to prevent relative vertical displacement at the interface	Lever Rule	Lever Rule	N/A
Open Steel Grid Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A
Concrete Deck on Multiple Steel Box Girders	b, c	Same as for Moment in Interior Beams		

Table 3-14 Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner

Type of Superstructure	Applicable Cross-Section (Table 3.6)	Correction Factor	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid or	a, e, k and also	$1.0 + 0.20\left(\frac{Lt_s^3}{K_g}\right)^{0.3} \tan\theta$	$0^o \le \theta \le 60^o$ $1100 \le S \le 4900$

Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- & Double T-Sections	i, j if sufficiently connected to act as a unit		$6000 \le L \le 73000$ $N_b \ge 4$
Cast-in-Place Concrete Multicell Box	d	$1.0+\left[0.25+\frac{L}{70d}\right]\tan\theta$	$0^o \le \theta \le 60^o$ $1800 \le S \le 4000$ $6000 \le L \le 73000$ $900 \le d \le 2700$ $N_c \ge 3$
Concrete Deck on Spread Box Beams	b, c	$1.0+\frac{\sqrt{Ld}}{6S}\tan\theta$	$0^o \le \theta \le 60^o$ $1800 \le S \le 3500$ $6000 \le L \le 43000$ $450 \le d \le 1700$ $N_b \ge 3$
Concrete Box Beams used in Multibeam Decks	f, g	$1.0+\frac{L\sqrt{\tan\theta}}{90d}$	$0^o \le \theta \le 60^o$ $6000 \le L \le 37000$ $430 \le d \le 1500$ $900 \le b \le 1500$ $5 \le N_b \le 20$

3.2.2 Dynamic Load Allowance: IM

The static effects of the design truck and tandem, except for pedestrian and lane load, shall be increased by the percentage specified in Table 3-15 for dynamic load allowance.

Table 3-15 Dynamic Load Allwance, IM

Component	Deck Joints – All Limit States	All Other Components	
		Fatigue and Fracture Limit State	All Other Limit States
IM	75 %	15 %	33 %

For buried structures such as culverts, IM shall be taken as:

$$IM = 33 * (1.0 - 4.1 * 10^{-4} * D_E) \geq 0\,\%$$

Where, D_E is the minimum depth of earth cover above the structure in mm.

3.2.3 Centrifugal Forces: CE

Centrifugal forces, which is to be applied horizontally at a distance 1800 mm above the roadway surface, shall be taken as the product of the axle weights of the design truck or tandem, multiple presence factors and the factor C, taken as:

$$C = \frac{4}{3}\frac{v^2}{gR}$$

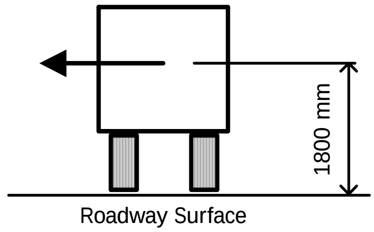


Figure 3-5 Centrifugal Force

where:

v = highway design speed (m/s) ; shall not be taken to be less than the value specified in the Specification for Road Design
g = gravitational acceleration: 9.807 (m/s^2)
R = radius of curvature of traffic lane (m)

3.2.4 Braking Force: BR

The braking force shall be taken as the greater of:

1) 0.25 * Design Truck
2) 0.25 * Design Tandem
3) 0.05 * (Design Truck + Design Lane)
4) 0.05 * (Design Tandem + Design Lane)

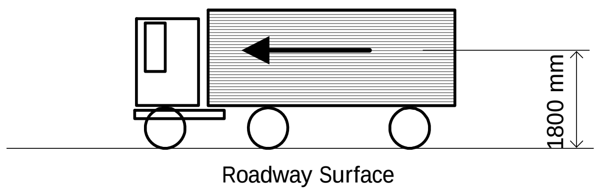


Figure 3-6 Braking Force

These forces shall be assumed to act horizontally at a distance of 1800 mm above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.
Multiple presence factors shall apply.

3.2.5 Vehicular Collision Force: CT

Unless protected as followings, abutments and piers located within a distance of 9000 mm to the edge of roadway, shall be designed for an equivalent static force of 1,800 kN, which is assumed to act in any direction in a horizontal plane, at a distance of 1200 mm above ground.

1) By an embankment;

2) By a structurally independent, crashworthy ground-mounted 1370 mm high barrier, located within 3000 mm from the component being protected; or

3) By a 1070 mm high barrier, located at more than 3000 mm from the component being protected

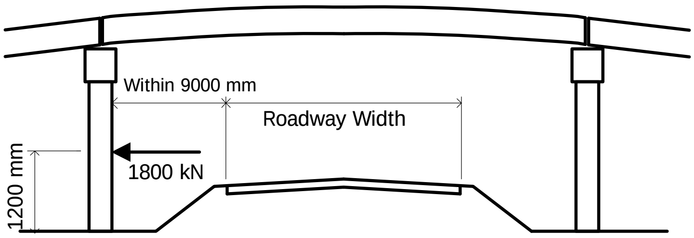


Figure 3-7 Vehicle Collision Force

3.3 Water Loads: WA

For water loads, static pressure, buoyancy and stream pressure shall be considered together with the effect due to scour such as riverbed elevation change at strength and service limit state.

3.3.1 Static Pressure (p_w)

Static pressure of water shall be assumed to act perpendicular to the surface that is retaining water. Pressure shall be calculated as the product of height (Z) of water above the point of consideration, the density of water (γ_w), and g (the acceleration of gravity).

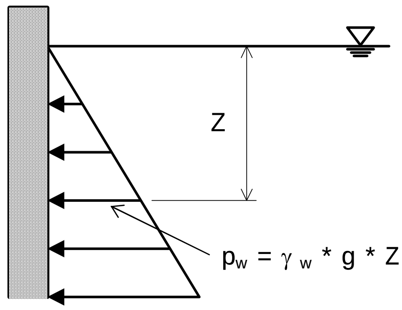


Figure 3-8 Static Water Pressure

3.3.2 Buoyancy

Buoyancy shall be considered to be an uplift force, taken as the sum of the vertical components of static pressures acting on all components below the design water level.

3.3.3 Stream Pressure

(1) Longitudinal

The pressure of flowing water acting in the longitudinal direction of substructure shall be taken as:

$$p = 5.14 * 10^{-4} C_D V^2$$

where:

p = pressure of flowing water (MPa)

C_D = drag coefficient for piers as specificed in Table 3-16

V = design velocity of water for the design flood in strength and service limit states and for the check flood in the extreme event limit state (m/s)

Table 3-16 Drag Coefficient

Type	C_D
Semicircular-nosed Pier	0.70
Square-ended Pier	1.40
Debris lodges against the Pier	1.40
Wedge-nosed Pier with Nose Angle 90° or less	0.80

(2) Transverse

The lateral, uniformly distributed pressure on a substructure due to water flowing at an angle, θ, to the longitudinal axis of the pier shall be taken as:

$$p = 5.14 * 10^{-4} C_L V^2$$

where:

p = lateral pressure (MPa)

C_L = lateral drag coefficient specificed in Table 3-17

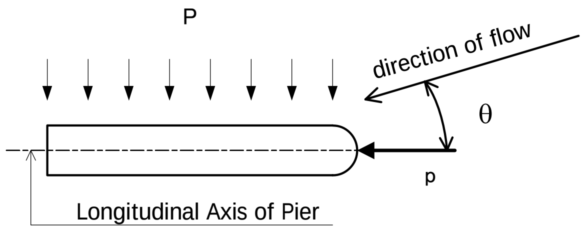


Figure 3-9 Transverse Water Pressure

Table 3-17 Lateral Drag Coefficient

Angle, θ, between direction of flow and longitudinal axis of the pier	C_L
0 °	0.00
5 °	0.50
10 °	0.70
20 °	0.90
≥ 30 °	1.00

3.4 Wind Load

3.4.1 Horizontal Wind Load

This Article provides design horizontal wind loads for conventional bridge structures. For long span or wind-sensitive structures, such as suspension or cable-stayed bridges, specific wind climate studies and wind tunnel tests should be carried out to determine the design wind effects.

Pressures specified herein shall be assumed to be caused by a base design wind velocity, V_B of 160km/hr.

Wind load shall be assumed to be uniformly distributed on the area exposed to the wind. The exposed area shall be the sum of areas of all component, including floor system and railing, as seen in elevation taken pependicular to the assumed wind direction. This direction shall be varied to determine the extreme force effect in the structure or in its components. Areas that do not contribute to the extreme force effect under cosideration may be neglected in the analysis.

For bridge or parts of bridges more than 10000 mm above low ground or water level, the design wind velocity, V_{DZ}, should be adjusted according to:

$V_{ZD} = 2.5\ Vo\ (V_{10}/V_B)\ln(Z/Z_0)$

Where:

V_{ZD}: design wind velocity at design elevation, Z (km/h)

V_{10} = wind velocity at 10 000 mm above low ground or above design water level (km/hr.)

V_B = base wind velocity of 160 km/hr. at 10 000 mm height, yielding design pressures specified in

Articles 3.8.1.2 and 3.8.2

Z = height of structure at which wind loads are being calcul ated as measured from low ground, or from water level, > 10 000 mm

V_0= friction velocity, a meteorological wind characteristic taken, asspecified in Table 3-18, for various up wind surface characteristic (km/hr .)

Z_o = friction length of upstream fetch, a meteorological wind charracteristic taken as specified in Table 3-18 (mm)

Table 3-18 Values of V_0 and Z_0 for Various Upstream Surface Conditions.

Condition	Open Country	Sub Urban	City
V_o(km/hr)	13.2	17.6	19.3
Z_o	70	1000	2500

V_{10} may be established from:

- Basic Wind Speed chartsavailable i n A SCE 7- 88 for various recurrence intervals,
- Site-specific wind surveys, and

- I n the absence of better criterion, the assumpti on that $V_{10}= V_B= 160$ km/hr.

3.5 Earthquake Effects: EQ

3.5.1 General

Earthquake loads shall be taken to be horizontal force effects for superstructures, substructures, foundations and connections between superstructures and substructures. These loads are determined based on the following items and seismic design procedure is shown in Figure 3-11.

1) Acceleration Coefficient (AC) at bridge site
2) Importance Categories(IC) for each bridge
3) Seismic Zone based on AC for each bridge
4) Site Effects (S) based on soil profile type
5) Period of Vibration of the m^th mode (T_m) for structure

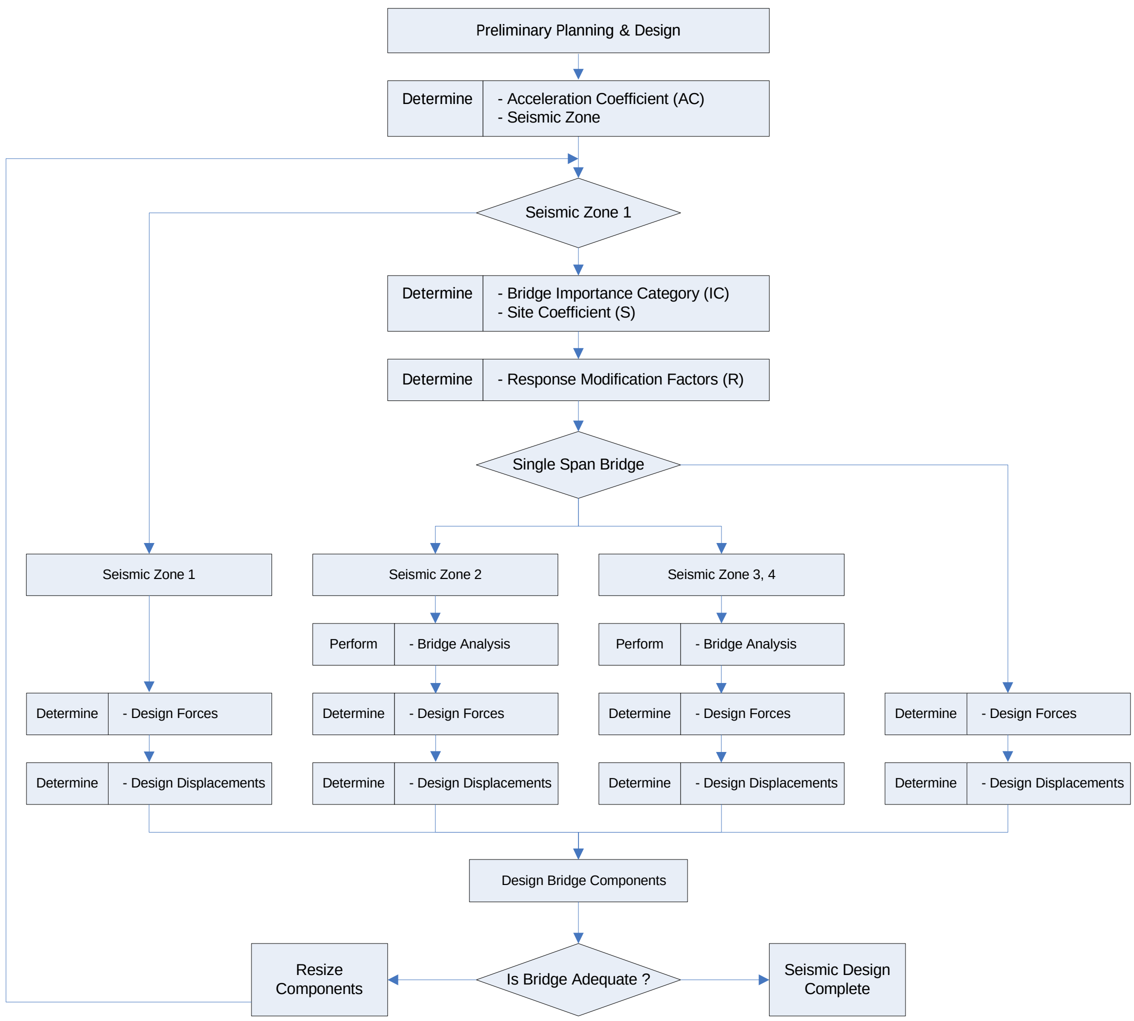


Figure 3-10 Seismic Design Procedure

This Article shall apply to structures such as bridges and culverts under the following conditions:

1) Bridges shall be of conventional slab, beam girder, box girder and truss superstructure construction with spans not exceeding 150 m.
2) Seismic effect shall not be considered for completely buried structures, except where they cross active faults.

3.5.2 Acceleration Coefficients (AC) and Seismic Zone

Each bridge shall be assigned to one of the four seismic zones in accordance with Table 3-19

Table 3-19 Seismic Zones

Acceleration Coefficient	Seismic Zone	MSK – 64 class
$A \leq 0.09$	1	Class ≤ 6.5
$0.09 < A \leq 0.19$	2	6.5 < Class ≤ 7.5
$0.19 < A \leq 0.29$	3	7.5 < Class ≤ 8

3.5.3 Importance Categories

The bridges shall be categorized into one of the following three importance categories. For project bridges, although they are located on National Highway No.13, which is one of the most important road in Lao and shall be open to traffic at any time, expected seismic force is not large.

Considering the expected small size of seismic force other than their importance, project bridges are categorized as “Essential Bridges”.

Table 3-20 Importance Categories

Critical Bridges	:	To remain open to all traffic after the design earthquake and be usable by emergency vehicles and for security/defense purposes immediately a large earthquake, e.g., a 2500 year return period event.
Essential Bridges	:	As a minimum, to be open to emergency vehicles and for security/defense purposes immediately after the design earthquake, i.e., a 475-year return period event.
Other Bridges	:	Others.

3.5.4 Site Effects

Based on the soil profile at each bridge site, site effects shall be included in the determination of seismic loads for bridges. The site coefficients S are shown in Table 3-21 with their soil conditions.

Table 3-21 Site Coefficients

Site Coefficient	Soil Profile Type			
	I	II	III	IV
S	1.00	1.20	1.50	2.00
Soil Conditions	Rock of any description, either shale-like or crystalline in nature, or Stiff soils where soil depth is less than 60 m, and soil types overlying rock are stable deposits of sands, gravels, or stiff clays	Stiff cohesive or deep cohesionless soils where soil depth exceeds 60 m and soil types overlaying the rock are stable deposits of sands, gravels, or stiff clays	Soft to medium-stiff clays and sands, characterized by 9 m or more of soft to medium-stiff clays	Soft clays or silts greater than 12 m in depth

3.5.5 Elastic Seismic Response Coefficient

Although the ground condition is expected to be poor, the extent of seismic force may be quite small even with the amplification of structural movement since acceleration coefficient is small.

The followings seismic force shall apply to bridges of conventional slab, beam girder, box girder, and truss superstructure construction with spans not exceeding 150 m. For other types of construction and bridges with spans exceeding 150 m, appropriate studies should be made.

$$H = C_{sm} \times W$$

The elastic seismic response coefficient, C_{sm} for the mth mode of vibration shall be taken as:

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A$$

Where:

T_m = period of vibration of the mth mode (s); based on the nominal unfactored mass of the component or structure.

A = acceleration coefficient

S = site coefficient

The following exceptions shall also be considered:

Exception-1	$C_{sm} \leq 2.0A$	For bridges on soil profile III or IV, and in areas where the coefficient A is not less than 0.30
Exception-2	$C_{sm} = A(0.8 + 4.0T_m)$	For bridges on soil profile III or IV, and for modes other than the fundamental mode that have periods less than 0.3 s
Exception-3	$C_{sm} = \frac{3AS}{T_m^{\frac{4}{3}}}$	For period of vibration for any mode exceeds 4.0 s

Seismic effects for box culverts and buried structures need not be considered, except where they cross active fault.

3.5.6 Response Modification Factors

Seismic design force effects for substructures and the connections between parts of structure, shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, R, specified in Tables 3-22 and 3-23 respectively.

Seismic design force effects = H / R

where:

H = elastic seismic force

R = response modification factor

Table 3-22 Response Modification Factors – Substructures

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers laregr dimension	1.5	1.5	2.0
Reinforced concrete pile bents - Vertical piles only - With batter piles	 1.5 1.5	 2.0 1.5	 3.0 2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents - Vertical piles only - With batter piles	1.5 1.5	3.5 2.0	5.0 3.0
Multiple column bents	1.5	3.5	5.0

Table 3-23 Response Modification Factors – Connections

Connection	All Important Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

In case an inelastic time history method of analysis is used, the response modification factors shall be taken as 1.0 for all substructure and connections.

3.5.7 Combination of Seismic Force Effects

Because of the directional uncertainty of earthquake motions, the following two load cases combining elastic member forces resulting from earthquakes in two perpendicular horizontal directions, which are usually longitudinal and transverse axes of the bridge, must be considered. For a curved bridge, longitudinal axes is often taken as the line joining the two abutments.

Load Case 1: $1.0\ F_L + 0.3\ F_T$

Load Case 2: $0.3\ F_L + 1.0\ F_T$

where:

F_L = absolute elastic member forces due to an earthquake in the direction of the lonfitudinal axis of the bridge

F_T = absolute elastic member forces due to an earthquake in the direction of the transverse axis of the bridge

3.5.8 Calculation of Design Forces

As the project bridges are located in Seismic Zone 1, no seismic analysis is required. However, provision of the following horizontal design minimum connection force shall be made to the connections.

Table 3-24 Minimum Design Horizontal Connection Force

Conditions	Design Horizontal Force
Acceleration coefficient, A, is less than 0.025 and soil profile is either Type-I or Type-II	<u>Longitudinal</u> 0.10 times the vertical reaction due to the tributary permanent load and tribtary live loads assumed to exist during earthquake. <u>Transverse</u> Vertical reaction shall be the permanent load reacting at the

	bearing.
All other sites	<u>Longitudinal</u> 0.20 times the vertical reaction due to the tributary permanent load and tribtary live loads assumed to exist during earthquake. <u>Transverse</u> Vertical reaction shall be the permanent load reacting at the bearing.

3.6 Earth Pressure: EH

Earth pressure depends on the type of soil, its water content and creep behavior, degree of compaction, location of groundwater table, soil-structure interaction, surcharge loads and dynamic effects.

For earth pressure coefficient, either Coulomb and Rankine wedge theory may be used. However, in general, Coulomb theory applies for gravity, semigravity and prefabricated modular walls with relatively steep back faces, and concrete cantilever walls with short heels, which are used for permanent bridges and road related structures. Therefore, the followings are based on Coulomb theory. For other structures such as nongravity cantilevered walls and temporary cofferdams, reference shall be made to Sections 3.11.5.5 – 3.11.5.7 of AASHTO LRFD 2007.

Lateral earth pressure shall be assumed to be linearly proportional to the depth of eath and taken as:

$$p = K * \gamma_s * g * z \ (*10^{-9})$$

where:

p = lateral earth pressure (MPa)

K = coefficient of lateral earth pressure

γ_s = density of soil (kg/m^3)

z = depth below the surface of earth (mm)

g = gravitational acceleration 9.807 (m/s^2)

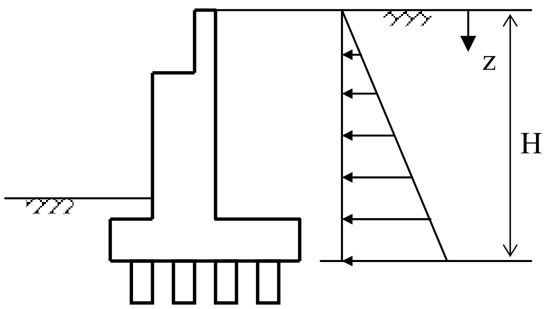


Figure 3-11 Earth Pressure

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of H/3 above the base of the wall, where H is the total wall height, measured from the surface of the ground at the back of the wall to the bottom of the footing or the top of the leveling pad (for MSE walls).

(1) At-Rest Lateral Earth Pressure Coefficient, Ko

$$Ko = 1 - \sin \phi'_f \quad \text{for normally consolidated soils}$$

$$= (1 - \sin \phi'_f)(OCR)^{\sin \phi'_f} \quad \text{for overconsolidated soils}$$

where:

ϕ'_f = effective friction angle of soil (deg)

Ko = coefficient of at-rest lateral earth pressure

OCR = overconsolidation ratio

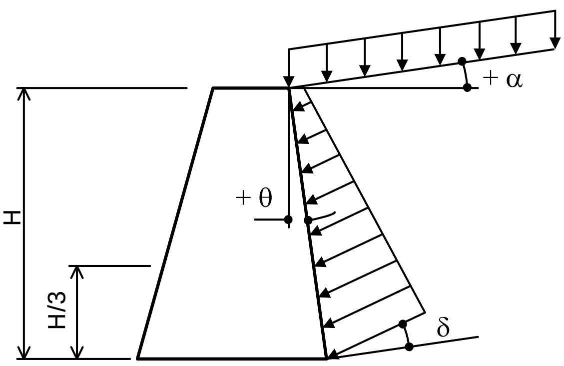
Table 3-25 Typical Coefficient of Lateral Earth Pressure At-Rest

Soil Type	Coefficient of Lateral Earth Pressure, Ko			
	OCR=1	OCR=2	OCR=5	OCR=10
Loose Sand	0.45	0.65	1.10	1.60
Medium Sand	0.40	0.60	1.05	1.55
Dense Sand	0.35	0.55	1.00	1.50
Silt (ML)	0.50	0.70	1.10	1.60
Lean Clay (CL)	0.60	0.80	1.20	1.65
Highly Plastic Clay (CH)	0.65	0.80	1.10	1.40

(2) Active Lateral Earth Pressure Coefficient, Ka

$$Ka = \frac{\cos^2(\phi - \theta)}{\Gamma_1 \left[\cos^2 \theta \cos(\theta + \delta)\right]}$$

$$\Gamma_1 = \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha)}{\cos(\theta + \delta)\cos(\theta - \alpha)}}\right]^2$$



where:

δ = friction angle between fill and wall (deg)

α = angle of fill to the horizontal (deg)

θ = angle of back face of wall to the vertical (deg)

ϕ = effective angle of internal friction (deg)

(3) Passive Lateral Earth Pressure Coefficient, Kp

For noncohesive soils, values of the coefficient of passive lateral earth pressure Kp may be taken from Figure 3.11.5.4-1 of Specification AASHTO LRFD 2007 for the case of a sloping or vertical wall with a horizontal backfill or from 3.11.5.4-2 of Specification AASHTO LRFD 2007 for the case of a vertical wall and sloping backfill.

For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory. When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

$$p_p = K_p * \gamma_s * g * z * 10^{-9} + 2c\sqrt{Kp}$$

where:

p_p = lateral earth pressure (MPa)
γ_s = density of soil (kg/m^3)
z = depth below the surface of earth (mm)
c = soil cohesion (MPa)
Kp = coefficient of passive lateral earth pressure
g = gravitational acceleration 9.807 (m/s^2)

(4) Seismic Active Earth Pressure Coefficient K_{ae}:

Seismic active earth pressure P_{ae} shall be taken as:

$$P_{ae} = \frac{1}{2} g\gamma H^2 (1-k_v) K_{ae} \times 10^{-9}$$

for which:

$$K_{ae} = \frac{\cos^2(\phi - \theta_o - \theta)}{\Gamma_2 \cos\theta_o \cos^2\theta \cos(\delta + \theta + \theta_o)}$$

$$\Gamma_2 = \left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta_o-\alpha)}{\cos(\delta+\theta+\theta_o)\cos(\alpha-\theta)}}\right]^2$$

where:

θ_o = arc tan (k_h/(1-k_v)) (deg)
k_h = horizontal acceleration coefficient
k_v = vertical acceleration coefficient

(5) Seismic Passive Earth Pressure Coefficient K_{pe}:

Seismic active earth pressure P_{ae} shall be taken as:

$$P_{pe} = \frac{1}{2} g\gamma H^2 (1-k_v) K_{pe} \times 10^{-9}$$

for which:

$$K_{pe} = \frac{\cos^2(\phi - \theta_o + \theta)}{\Gamma_3 \cos\theta_o \cos^2\theta \cos(\delta - \theta + \theta_o)}$$

$$\Gamma_3 = \left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta_o+\alpha)}{\cos(\delta-\theta+\theta_o)\cos(\alpha-\theta)}}\right]^2$$

where:

θ_o = arc tan (k_h/(1-k_v)) (deg)

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

3.7 Force Effects due to Superimposed Deformations: TU, TG, SH, CR, SE

3.7.1 Uniform Temperature: TU

Although average bridge temperature ranges are specified in Specification AASHTO LRFD 2007, **±15 deg.**

The coefficient of thermal expansion shall be taken as:

For normal density concrete:	$10.8 * 10^{-6}$ / ^{o}C
For low-density concrete:	$9.0 * 10^{-6}$ / ^{o}C
For all grades of structural steel:	$11.7 * 10^{-6}$ / ^{o}C

3.6.2 Temperature Gradient: TG

The effect of temperature difference within structure is a function of solar gain to the deck surface.

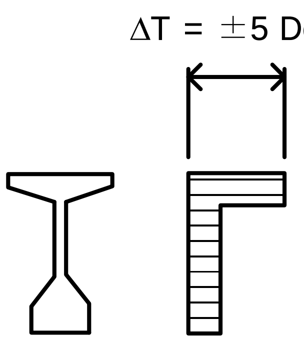


Figure 3-12 Gradient Temperature

3.6.3 Differential Shrinkage: SH

The effect of differential shrinkage shall be considered to determine the loss of prestressing force and deflection. For moist cured concretes devoid of shrinkage-prone aggregates, the strain due to shrinkage, □ sh, at time, t, is taken as:

$\varepsilon_{sh} = -k_s k_h \left(\frac{t}{35.0+t}\right) 0.51*10^{-3}$ for moist-cured concrete

$= -1.20 k_s k_h \left(\frac{t}{35.0+t}\right) 0.51*10^{-3}$ for moist-cured concrete is exposed to drying before 5 days of curing have elapsed

$= -k_s k_h \left(\frac{t}{55.0+t}\right) 0.56*10^{-3}$ for steam-cured concrete

where:

t = drying time (day)

k_s = size factor

$$= \left[\frac{\dfrac{t}{26e^{0.0142(V/S)}+t}}{\dfrac{t}{45+t}}\right]\left[\frac{1064-3.70(V/S)}{923}\right]$$

k_h = humidity factor

$= \frac{140-H}{70}$ for H < 80 %

$= \frac{3(100-H)}{70}$ for H ≥ 80 %

V/S = volume to surface area ratio ≤ 150 mm

3.7.4 Creep: CR

The effect of creep shall be considered to determine the loss of prestressing force and deflection. The creep coefficient is taken as:

$$\psi(t, t_i) = 3.5 k_c k_f \left(1.58 - \frac{H}{120}\right) t_i^{-0.118} \frac{(t-t_i)^{0.6}}{10.0+(t-t_i)^{0.6}}$$

for which:

$$k_f = \frac{62}{42 + f_c'}$$

where:

H = relative humidity (%)

k_c = factor for the effect of the volume-to-surface ratio of the component

$$= \left[\frac{\dfrac{t}{26e^{0.0142(V/S)}+t}}{\dfrac{t}{45+t}}\right]\left[\frac{1.80+1.77e^{-0.0213(V/S)}}{2.587}\right]$$

k_f = factor for the effect of concrete strength

t = maturity of concrete (day)

t_i = age of concrete when load is initially applied (day), In case accelerated curing by steam or radiant heat is used, such one day may be taken as equal to seven days of normal curing.

f_c' = specified compressive strength at 28 days (MPa)

V/S = volume to surface area ratio ≤ 150 mm, The surface area used in determining V/S should include only the area that is exposed to atmospheric drying. For poorly ventilated enclosed cells, only 50 % of the interior perimeter should be used in calculating the surface area.

3.7.5 Settlement: SE

For effects due to extreme values of differential settlements among substructures and within individual substructure units shall be considered.

The angular distortions between adjacent foundations greater than 0.008 in simple spans and 0.004 in continuous spans should not be permitted (C10.6.2.2.1 of AASHTO LRFD).

3.8 Friction Forces: FR

Forces due to friction on the sliding or rotating surface shall be considered.

Table 3-26 Friction Coefficient for Each Bearing Type

Friction Mechanism	Type of Bearing	Friction Coefficient
Rolling Friction (Steel)	Roller Bearing	0.05
	Rocker Bearing	0.05
Sliding Friction (Steel)	Cylindrical Bearing	0.25
	Spherical Bearing	0.25
	Pot Bearing	0.25
Shear Deformation of Rubber	Elastmeric Bearing	0.15

3.9 Vessel Collision: CV

3.9.1 Collision Force on Pier

Vessel collision force on pier is defined both due to ship and barge as described below. In this project, both of them must be considered to design substructures and foundations.

(1) Ship Collision Force on Pier

Head-on ship collision impact force on a pier is taken as:

$$P_s = 1.2 * 10^5 \; V \; DWT^{0.5}$$

where:

P_s = equivalent static vessel impact force (N)

DWT = deadweight tonnage of vessel (Mg)

V = vessel impact velocity (m/s)

(2) Barge Collision Force on Pier

The collision impact force on a pier for a river-going barge shall be taken as:

$$P_B = 6.0 * 10^4 a_B \qquad a_B < 100 \text{ mm}$$

$$= 6.0 * 10^6 + 1600 * a_B \qquad a_B \geq 100 \text{ mm}$$

where:

P_B = equivalent static barge impact force (N)

a_B = barge bow damage length (mm)

$$= 3100\left(\sqrt{1+1.3*10^{-7}KE} - 1\right)$$

where:

KE = vessel collision energy (joule)

$= 500 C_H M V^2$

where:

C_H = hydrodynamic mass coefficient

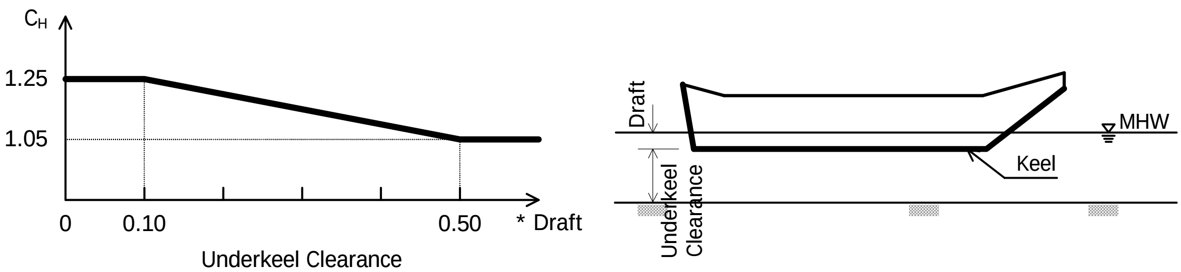


Figure 3-13 Hydrodynamic Mass Coefficient and Underkeel Clearance

M = vessel displacement tonnage (Mg) = DWT

V = vessel impact velocity (m/s)

Vs = mean annual stream velocity adjacent to the bridge element under consideration (m/s)

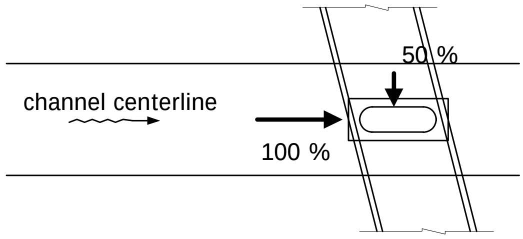
Theabove design impact velocity is defined as shown in Table 3-29.

Table 3-29 Design Impact Velocity for Design Vessels

Design Vessel	Design Impact Velocity, V (m/s)
Self-propelled Vessel ≥ 1000 DWT	3.3 + Vs
Self-propelled Vessel < 1000 DWT	2.5 + Vs
Towed Barge	1.6 + Vs

3.9.3 Application of Impact Forces

(1) Substructure Design



For substructure design, equivalent static forces, parallel and normal to the centerline of navigable channel, shall be separately applied as follows:

1) 100 % of the design impact force in a direction parallel to the centerline of navigable channel, or
2) 50 % of the design impact force in a direction normal to the centerline of navigable channel

The impact force shall be applied to a substructure in accordance with the following criteria:

* For overall stability, design impact force is applied as a concentrated force on the substructure at the mean annual high water level of the waterway, as shown in Figure 3-17.

* For local collision forces, design impact force is applied as a vertical line load equally distributed on the depth of the head block (H_L), as shown in Figure 3-18.

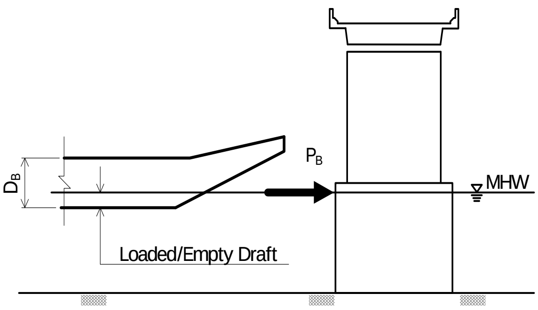


Figure 3-14 Load Application for Overall Stability

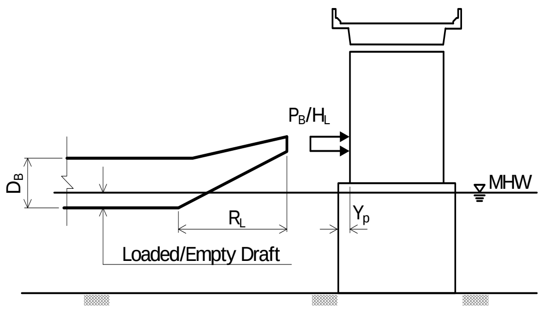


Figure 3-15 Load Application for Local Collision

(2) Superstructure

For superstructure design, design impact force shall be applied as an equivalent static force transverse to the superstructure component in a direction parallel to the centerline of navigable channel.

4. Considerations of Prestressing Force

After a reinforced concrete member is precompressed by prestressing tendons, reduction of effectiveness of prestressing force will occur. Some of the losses occur almost instantaneously while others take years before they finally dampen out. These effects shall be considered in design of prestressed reinforced concrete.

The followings are the summary and details of prestressing loss Δf_{pT} for both pretensioned and posttensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR}$$ For pretensioned members

$$\Delta f_{pT} = \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pF} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR}$$ For posttensioned members

	Loss of Prestress		
Instantaneous Losses	Δf_{pA}	:	Due to slip of the tendons in the anchorages for only posttensioned members
	Δf_{pES}	:	Due to elastic compression (shortening) of the concrete
	Δf_{pF}	:	Due to friction between a tenson and its conduit
Long-time Losses	Δf_{pSR}	:	Due to shrinkage of concrete
	Δf_{pCR}	:	Due to creep of concrete
	Δf_{pR}	:	Due to relaxation of the prestressing tendon

(1) Loss due to Friction : Δf_{pF}

Losses due to friction between the internal prestressing tendons and the duct wall may be taken as:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx+\mu\alpha)}\right)$$

where:

f_{pj} = stress in the prestressing steel at jacking (MPa)

x = length of a prestressing tendon from the jacking end to any point under consideration (mm)

K = wobble friction coefficient (per mm of tendon)

μ = coefficient of curvature friction (per radian)

α = sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation (rad).

Table 4-1. Friction Coefficient

Type of Steel	Type of Duct	K [1]	μ
Wire or Strand	Rigid and semirigid galvanized metal sheathing	$4.0 * 10^{-6}$	0.25
	Polyethylene	$4.0 * 10^{-6}$	0.23
	Rigid steel pipe deviators for external tendons	$4.0 * 10^{-6}$	0.25
High Strength Bars	Galvanized metal sheathing	$4.0 * 10^{-6}$	0.30

(2) Loss due to Anchorage Set : Δf_{pA}

Although no specification on loss due to anchorage set is given both in the Specification AASHTO LRFD 2007 and AASHTO LRFD, it shall be considered in effectiveness of prestressing force for posttensioned member. It shall be taken as:

$$\Delta l = \frac{A_{EP}}{A_p E_p}$$

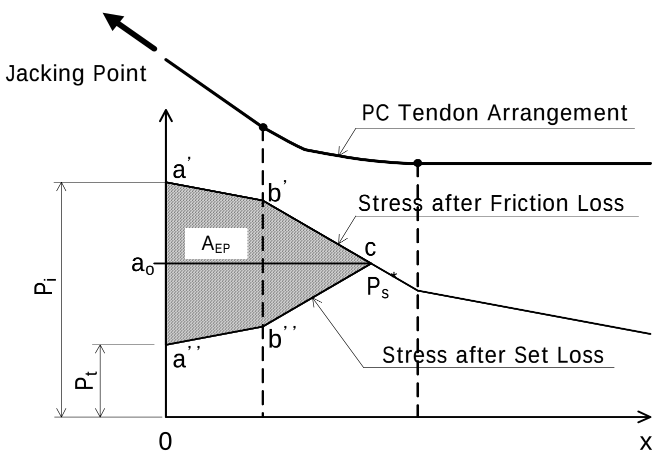
where:

Δl = Set of prestressing steel (mm)

A_{EP} = Area covered by a'b'cb''a''

A_P = Area of prestressing steel (mm^2)

E_P = Modulus of Elasticity of prestressing steel (MPa)



FFigure 4-1 Loss due to Anchorage Set

This equation is based on the supposition that the friction coefficient in times of jacking and releasing are same, as a result of which the line a'b'c is symmetrical with the line a''b''c with respect to the line a_oc. The point "c" is determined by that given set of prestressing steel is equal to the value of area of A_{EP} divided by A_pE_p.

Set of prestressing steel will be given from the manufacturer.

1 Even though wobble friction coefficient of $6.6 * 10^{-7}$ is given in AASHTO LRFD, $4.0 * 10^{-6}$ will be used in this project from the previous practices in Lao.

Table 4-2. Set of Prestressing Steel by Anchorage Type

Anchorage Type	Set (mm)	Anchorage Type	Set (mm)
12 ϕ 5	4	12T15.2 M294	11
12 ϕ 7	5	12T15.2 M319	11
12 ϕ 8	6	1T13	3
12T12.4 M199	11	1T15	3
12T12.7 M199	12	1T18	3
12T12.4 M220	7	1T19	3.5
12T12.7 M220	8	1T22	4

(3) Loss due to Elastic Shortening : Δf_{pES}

Losses due to elastic shortening in pretensioned and posttensioned member shall be respectively taken as:

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + e_m{}^2 A_g\right) - e_m M_g A_g}{A_{ps}\left(I_g + e_m{}^2 A_g\right) + \dfrac{A_g I_g E_{ci}}{E_p}}$$

For pretensioned members

$$\Delta f_{pES} = \frac{N-1}{2N} \frac{A_{ps} f_{pbt}\left(I_g + e_m{}^2 A_g\right) - e_m M_g A_g}{A_{ps}\left(I_g + e_m{}^2 A_g\right) + \dfrac{A_g I_g E_{ci}}{E_p}}$$

For posttensioned members

where:

f_{ps} = are of prestressing steel (mm^2)
A_g = gross area of section (mm^2)
E_{ci} = modulus of elasticity of concrete at transfer (MPa)
E_p = modulus of elasticity of prestressing tendons (MPa)
e_m = average eccentricity at midspan (mm)
f_{pbt} = stress in prestressing steel immediately prior to transfer (MPa)
I_g = moment of inertia of the gross concrete section (mm^4)
M_g = midspan moment due to member self-weight (N.mm)
N = number of identical prestressing tendons
f_{pj} = stress in the prestressing steel at jacking (MPa)

For posttensioned structures with bonded tendons, it may be calculated at the section of maximum moment.

For posttensioned structures with unbonded tendons, it can be calculated using the eccentricity of the prestressing steel averaged along the length of the member.

For slab system, it may be taken as 25 % of the obtained values from the above equation.

(4) Loss due to shrinkage : Δf_{pSR}

Loss due to shrinkage shall be taken as:

$$\Delta f_{pSR} = (117 - 1.03H) \text{ (MPa)}$$ For pretensioned member

$$\Delta f_{pSR} = (93 - 0.85H) \text{ (MPa)}$$ For posttensioned member

where:

H = average annual ambient relative humidity (%) = **75 %** to be used

(5) Loss due to creep : Δf_{pCR}

Loss due to creep shall be taken as:

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp} \geq 0$$

where:

f_{cgp} = concrete stress at center of gravity of prestressing steel at transfer (MPa)

Δf_{cdp} = change in concrete stress at center of gravity of prestressing steel due to permanent loads, with the exception of the load acting at the time the prestressing force is applied. This shall be calculated at the same section(s) for which f_{cgp} is calculated (MPa)

(6) Loss due to relaxation of prestressing tendons : Δf_{pR}

The total relaxation at any time after transfer shall be taken as the sum of the losses Δf_{pR1} and Δf_{pR2} specified below.

1) At transfer only in pretensioned members

$$\Delta f_{pR1} = \frac{\log(24.0t)}{10.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$ For stress-relieved strand

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$ For low-relaxation strand

where:

t = time estimated in days from stressing to transfer (days)

f_{pj} = initial stress in the tendon at the end of stressing (MPa)

f_{py} = specified yield strength of prestressing steel (MPa)

2) After transfer

For pretensioning with stress-relieved strands:

$$\Delta f_{pR2} = 138 - 0.4\Delta f_{pES} - 0.2\left(\Delta f_{pSR} + \Delta f_{pCR}\right) \quad \text{(MPa)}$$

For posttensioning with stress-relieved strands:

$$\Delta f_{pR2} = 138 - 0.3\Delta f_{pF} - 0.4\Delta f_{pES} - 0.2\left(\Delta f_{pSR} + \Delta f_{pCR}\right) \quad \text{(MPa)}$$

For prestressing steel with low-relaxation properties, 30 % of the above values shall be used.

For posttensioning with 1000 to 1100 MPa bars, the loss may be assumed to be 21 MPa.

where:

Δf_{pF} = friction loss below the level of 0.70 f pu at the point under consideration (MPa)

Δf_{pES} = loss due to elastic shortening (MPa)

Δf_{pSR} = loss due to shrinkage (MPa)

Δf_{pCR} = loss due to creep of concrete (MPa)

5. Materials

(1) Concrete

Although concrete strength for each structural element shall basically follow the Lao Standard considering local conditions, they may be modified based on the AASHTO LRFD for reasons of required properties. The followings are concrete strengths for each structural element to be used in this Project.

Table 5-1. Concrete Strength by Structural Member

Compressive Strength at 28 days (MPa) (Cylinder Specimen)	Structural Member	Remarks
50	Pretensioned Slab/Girder	During design period, these may be modified due to requirements.
45	Free Cantilever PC Girder	
40	Post-tensioned PC I-Girder Cast-in-situ PC Slab/Girder	
30	RC Girder Diaphragm (Crossbeam)	

	RC Deck Slab Substructure (Pier, Abutment, Pile Caps, Wingwall) Retaining Wall, Box Culvert Precast Reinforced Concrete Plate Precast Pile Precast Parapet	
25	Approach Slab Pipe Culvert Precast Concrete Curb	
30	Cast-in-situ Bored Pile	
18	Non-reinforced Concrete Structure Lean Concrete	

In this project, only normal density concrete shall be used. The properties of concrete are as shown below.

Table 5-2. Concrete Properties

Modulus of Elasticity (MPa)	Poisson's Ratio	Modulus of Rupture (MPa)
$E_c = 0.043\gamma_c^{1.5}\sqrt{f'_c} \quad (1440 \le \gamma_c \le 2500)$ γ_c = density of concrete (kg/m^3) f'_c = specified strength of concrete (MPa)	0.20	$f_r = 0.63\sqrt{f'_c}$ flexure tensile stress

Stress limits for concrete in Service Limit State in PC are shown in Tables 6-3, 6-4 and 6-5. For RC, as the width of flexure cracks is controlled by distributing the reinforcement over the region of maximum concrete tension, stress limit for concrete is not described.

As for prestressed concrete structure, only fully prestressed components are permitted in this project.

Table 5-3. Temporary Tensile Stress Limits in PC before Losses

Bridge Type	Location	Stress Limit (MPa)
Other Than Segmentally Constructed Bridges	* In precompressed tensile zone without bonded reinforcement	Not Applicable
	* In areas other than the precompressed tensile zone and without bonded reinforcement	$0.25\sqrt{f'_{ci}} \le 1.38$
	* In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5 fy, not to exceed 210 MPa.	$0.63\sqrt{f'_{ci}}$
	* For handling stresses in prestressed piles	$0.415\sqrt{f'_{ci}}$

Table 5-4. Compressive Stress Limits in PC at Service Limit State after Losses

Location	Stress Limit (MPa)
* In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	0.45 f'c
* In other than segmentally constructed bridges due to live load and one-half the sum of effective prestress and permanent loads	0.40 f'c
* Due to the sum of effective prestress, permanent loads, and transient loads and during shipping and handling	0.60 ϕ_w f'c

Table 5-5. Tensile Stress Limits in PC at Service Limit State after Losses

Bridge Type	Location	Stress Limit (MPa)
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, assuming uncracked sections	
	* For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	$0.50\sqrt{f'_c}$
	* For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	$0.25\sqrt{f'_c}$
	* For components with unbonded prestressing tendons	No tension

(2) Reinforcing Bar

Two types of Grade 300 and Grade 420 shall be used. The properties and strength are as shown below.

Table 5-6. Properties and Stress Limit of Reinforcing Bars

Type	Yield Strength f_y (MPa)	Tensile Strength f_u (MPa)	Modulus of Elasticity (MPa)
Grade 300	300	500	200,000
Grade 420	420	620	200,000

(3) Prestressing Steel

Uncoated, stress-relieved or low-relaxation, seven-wire strand, or uncoated plain or deformed, high-strength bars, shall have the following properties and strength as shown in Table 5-7.

Table 5-7. Properties of Prestressing Strand and Bar

Material	Grade or Type	Diameter (mm)	Tensile Strength f_{pu} (MPa)	Modulus of Elasticity E_p (MPa)	Yield Strength f_{py} (MPa)
Strand	1725 MPa (Grade 250)	6.35 – 15.24	1725	197,000	0.85 f_{pu} for stress-relieved 0.90 f_{pu} for low-relaxation
	1860 MPa (Grade 270)	9.53 – 15.24	1860		0.90 f_{pu}
Bar	Type 1, Plain	19 – 35	1035	207,000	0.85 f_{pu}
	Type 2, Deformed	16 – 35	1035		0.80 f_{pu}

Stress limits for each tendon type are as shown in Table 5-8.

Table 5-8. Stress Limits for Prestressing Tendons

	Tendon Type		
	Stress-relieved Strand / Plain high-strength bars	Low Relaxation Strand	Deformed High-strength Bars
Pretensioning			
Immediately prior to transfer ($f_{pt} + \Delta f_{pES}$)	0.70 f_{pu}	0.75 f_{pu}	-
At service limit state after all losses (f_{pe})	0.80 f_{py}	0.80 f_{py}	0.80 f_{py}
Post-tensioning			
Prior to seating – short-term f_s may be allowed	0.90 f_{py}	0.90 f_{py}	0.90 f_{py}
At anchorages and couplers immediately after anchor set ($f_{pt} + \Delta f_{pES} + \Delta f_{pA}$)	0.70 f_{pu}	0.70 f_{pu}	0.70 f_{pu}
At end of the seating loss zone immediately after anchor set ($f_{pt} + \Delta f_{pES} + \Delta f_{pA}$)	0.70 f_{pu}	0.74 f_{pu}	0.70 f_{pu}
At service limit state after all losses (f_{pe})	0.80 f_{py}	0.80 f_{py}	0.80 f_{py}

6. Foundation Design

These bridges are located in the soft ground area, which bearing stratumn is 40 to 60 m deep from the ground surface. Therefore, the pile foundaton shall be adopted for foundation type.

6.1 Pile foundation

Pile foundation design shall be made for service limit states, strength limit states and extreme event limit states respectively. Each limit state include the followings:

	Verification Items	Remark
Service Limit States	Adequate Bearing resistance	Aollable Bearing Resistance
	Structural Resistance	Control of Cracking
	Tolerable Settlement	Considered Bridge Performance
	Tolerable Horizontal Displacement	
Strength Limit States	Adequate Bearing Resistance	Considered punching failure
	Structural Resistance	
	Horizontal Displacement	P-Y curve
Extreme Event Limit States	Bearing Resistance	
	Structural Resistance	
Service Limit state	Overall stability	Considered Lateral Flow

6.2 Driven Pile

6.2.1 Minimum Pile Penetration

Unless refusal is encountered, the following minimum pile penetration shall be made:

Pile Type	Ground Condition	Minimum Pile Penetration
Any pile type	Hard cohesive or dense granular material	Not less than 3,000 mm
	Soft cohesive or loose granular material	Not less than 6,000 mm
	Soft or loose upper stratum overlying a hard or firm stratum	Penetration shall be made to hard stratum by a distance sufficient to limit movement of the piles and attain sufficient bearing capacities
	Embankment Fill	A minimum of 3,000 mm through original ground
Piles for trestle or pile bents	Any condiiton	Distance equal to at least one-third the unsupported pile length

6.2.2 Pile Spacing, Clearances and Embedment

Figure 6-1 shows the minimum center-to-center pile spacings, distance from the side of any pile to the nearest edge of the footing and embedment length to footing in two cases pile top is projected after all damaged pile material has been removed and the pile top is attached to the footing by embedded bars or strands.

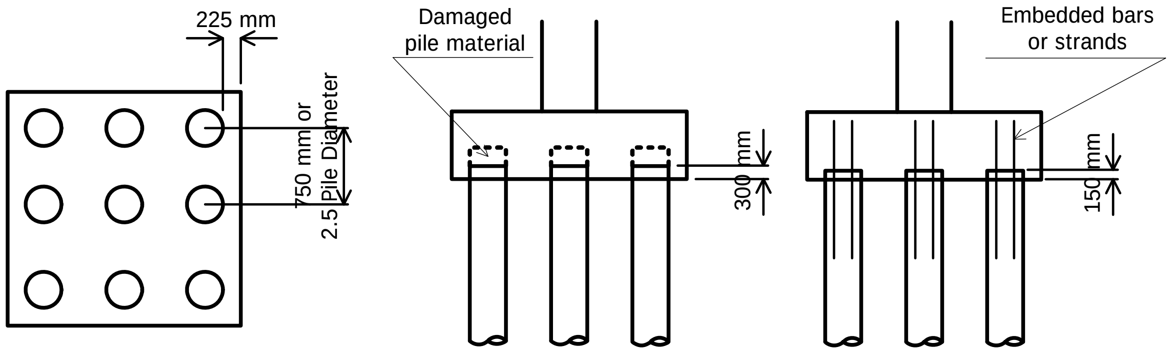


Figure 6-1 Minimum Pile Spacing, Clearances and Embedment of Pile Top to Footing

6.2.3 Service Limit State

6.2.3.1 Settlement of Pile Group (Service I Load Combination)

If the friction pile is adopted for foundation type, the following items should be examined.

For purposes of calculating settlements of pile group, loads shall be assumed to act on an equivalent footing located at two-thirds of the depth of embedment of the pile into the layer that provides support as shown in Figure 6-2. Vertical loads do not include the weight of piles or the soil between piles.

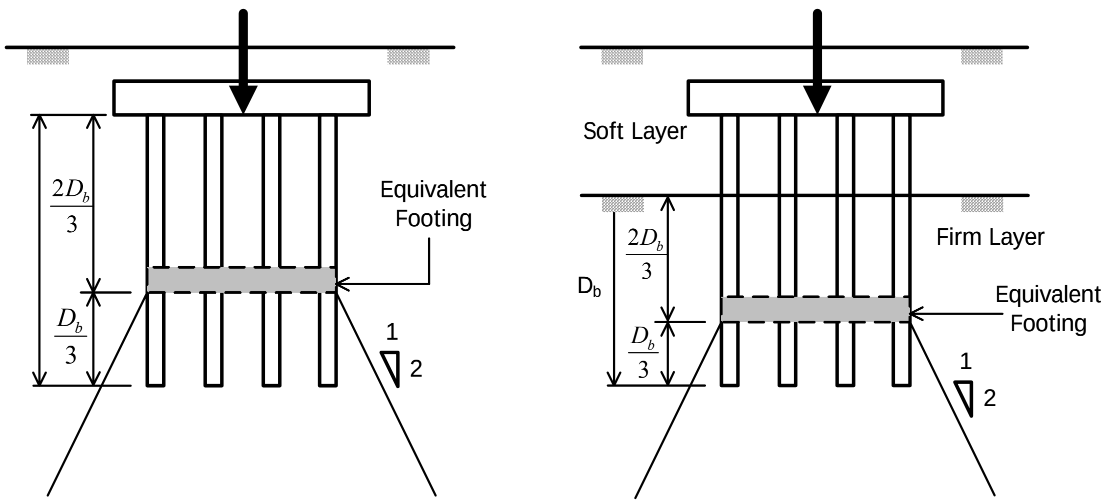


Figure67-2 Location of Equivalent Footing

Cohesiive Soil

Procedures used for shallow foundations shall be used to estimated the settlement of a pile group, using the equivalent footing location specified in Fig 6-4.

Cohesionless Soil

Using SPT: $$\rho = \frac{360ql\sqrt{X}}{N_{corr}}$$

Using CPT: $$\rho = \frac{qXl}{2q_c}$$

for which:

$$I = 1 - 0.125\frac{D'}{X} \geq 0.50$$

$$N_{corr} = \left[0.77\log_{10}\left(\frac{1.92}{\sigma_v'}\right)\right]N$$

where:

q = net foundation pressure applied at the bottom of equivalent footing; this pressure is equal to the applied load at the top of pile group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles (MPa)

X = width or smaller dimension of pile group (mm)

ρ = settlement of pile group (mm)

I = influence factor of the effective group embedment

D' = effective depth taken as $2D_b / 3$ (mm)

D_b = depth of embedment of piles in layer that provides support (mm)

N_{corr} = representative average corrected for overburden SPT blow count over a depth X below the equivalent footing (Blows / 300mm)

N = measured SPT blow count within the seat of settlement (Blows / 300mm)

σ_v' = effective vertical stress (MPa)

q_c = average static cone penetration over a depth X below the equivalent footing (MPa)

6.2.3.2 Horizontal Displacement

Design horizontal movement should not exceed 15 mm. \

6.2.4 Resistance at Strength Limit State

The resistances that shall be considered include bearing resistance of piles (shaft resistance and tip resistance), uplift resistance of piles, punching of piles through strong soil into a weaker layers and structural resistance of the piles.

6.2.4.1 Bearing Resistance of Single Pile

The factored bearing resistance of piles, Q_R, is taken as:

$$Q_R = \varphi Q_n = \varphi_{qp} Q_p + \varphi_{qs} Q_s$$

for which:

$$Q_p = q_p A_p, \text{ and } \quad Q_s = q_s A_s$$

where:

φ_p = resistance factor for the bearing resistance of a single pile
Q_p = pile tip resistance (N)
Q_s = pile shaft resistance (N)
q_p = unit tip resistance of pile (MPa)
q_s = unit shaft resistance of pile (MPa)
A_s = surface area of pile shaft (mm^2)
A_p = area of pile tip (mm^2)
φ_{qp} = resistance factor for tip resistance
φ_{qs} = resistance factor for shaft resistance

Table 6-1 Resistance Factors for Geotechnical Strength Limit State in Axially Loaded Piles

Methods/Soil/Condition		Resistance Factor
Ultimate Bearing Resistance of Single Piles	Skin Friction: Clay α – method (Tomlinson 1987) β – method (Esrig & Kirby 1979 and Nordlung method aplied to cohesive soils) λ – method (Vijayvergiya & Focht 1972)	 $0.70\ \lambda_v$ $0.50\ \lambda_v$ $0.55\ \lambda_v$
	End Bearing: Clay and Rock Clay (Skempton 1951) Rock (Canadian Geotechnical Society 1985)	 $0.70\ \lambda_v$ $0.50\ \lambda_v$
	Skin Friction and End Bearing: Sand SPT – method CPT – method	 $0.45\ \lambda_v$ $0.55\ \lambda_v$
	Wave equation analysis with assumed driving resistance	$0.65\ \lambda_v$

	Load Test	0.80 λ_v
Block Failure	Clay	0.65
Uplift Resistance of Single Piles	α – method β – method λ – method SPT – method CPT – method Load Test	0.60 0.40 0.45 0.35 0.45 0.80
Group Uplift Resistance	Sand Clay	0.55 0.55

Table 6-2 Values of λ_v

Method of controlling installation of piles and verifying their capacity during or after driving to be specified in the contract documents	Value of λ_v
Pile Driving Formulas, e.g., ENR, equation without stress wave measurements during driving	0.80
Bearing graph from wave equation analysis without stress wave measurements during driving	0.85
Stress wave measurements on 2% to 5% of piles, capacity verified by simplified methods, e.g., the pile driving analyzer	0.90
Stress wave measurements on 2% to 5% of piles, capacity verified by simplified methods, e.g., the pile driving analyzer and static load test to verify capacity	1.00
Stress wave measurements on 2% to 5% of piles, capacity verified by simplified methods, e.g., the pile driving analyzer and CAPWAP analyses to verify capacity	1.00
Stress wave measurements on 10% to 70% of piles, capacity verified by simplified methods, e.g., the pile driving analyzer	1.00

For calculating bearing resistance of piles, both semiempirical estimates such as total stress (α - method) or effective stress (β - method) methods and in-situ tests based estimates using SPT (Standard Penetration Test) or CPT (Cone Penetration Test) results are available.

In this project, for bridge foundations, SPT at site and undrained shear strength from laboratory test results are available. Therefore, semiempirical estimates using undrained shear strength S_u and in-situ tests based estimates using SPT are introduced.

1) Semiempirical Estimates

Shaft Resistance by Total Stress (α – Method) for Cohesive Soil

The nominal unit skin friction q_s is related to the effective stresses in the ground as:

$$q_s = \alpha S_u \quad \text{(MPa)}$$

where:

S_u = mean undrained shear strength (MPa)

α = adhesion factor applied to taken from Figure 10.6.3.3.2a-1 of AASHTO LRFD 2007

Shaft Resistance by Effective Stress (β – Method) for Cohesionless Soil

The nominal unit skin friction q_s is related to the effective stresses in the ground as:

$$q_s = \beta \sigma_v' \quad \text{(MPa)}$$

where:

σ_v' = vertical effective stress (MPa)

β = a factor taken from Figure 10.6.3.3.2b-1 of AASHTO LRFD 2007

Tip Resistance for Cohesive Soil

The nominal unit tip resistance of piles in saturated clay is taken as:

$$q_p = 9 S_u \quad \text{(MPa)}$$

where:

S_u = undrained shear strength of the clay near the pile tip (MPa)

2) In-situ Test Based Estimates using SPT

Tip Resistance for Cohesionless Soil

The nominal unit tip resistance for piles driven to a depth D_b into a cohesionless soil stratum is taken as:

$$q_d = \frac{0.038 N_{corr} D_b}{D} \leq q_l$$

for which:

$$N_{corr} = \left[0.77 \log_{10}\left(\frac{1.92}{\sigma_v'}\right)\right] N$$

where:

N_{corr} = representative SPT blow count near the pile tip corrected for overburden pressure, σ_v' (Blows / 300mm)

N = measured SPT blow count (Blows / 300mm)

D' = pile width or diameter (mm)

D_b = depth of embedment of piles in layer that provides support (mm)

q_l = limiting point resistance taken as 0.4 N_{corr} for sands and 0.3 N_{corr} for nonplastic silt (MPa)

Shaft Resistance for Cohesionless Soil

The nominal shaft resistance of piles in cohesionless soils is taken as:

For driven displacement piles[2]:

$$q_s = 0.0019\overline{N}$$

For non-displacement piles[3]:

$$q_s = 0.00096\overline{N}$$

where:

q_s = unit shaft resistance for driven piles (MPa)

$\overline{N}$ = average (uncorrected) SPT blow count along the pile shaft (Blows / 300mm)

6.2.3.2 Uplift

Uplift shall be considered when the force effects are tensile. When piles are subjected to uplift, they should be investigated for both resistance to pullout and structural ability to resist tension and transmit it to the footing.

A single pile factored uplift resistance shall be taken as:

$$Q_R = \varphi Q_n = \varphi_u Q_{us}$$

where:

φ_{ug} = resistance factor for uplift capacity specified in Table 6-1

Q_{us} = nominal uplift capacity due to shaft resistance (N)

Pile group factored uplift resistance shall be taken as:

$$Q_R = \varphi Q_n = \varphi_{ug} Q_{ug}$$

where:

φ_{ug} = resistance factor

Q_{ug} = nominal uplift resistance of the group (N)

The uplift resistance, Q_{ug}, of a pile group shall be taken as the lesser of:

2 Displacement piles, which have solid sections or hollow sections with a closed end, displace a relatively large volume of soil during penetration

3 Non-displacement piles usually have relatively small cross-sectional areas, e.g., steel-H piles and open ended pipe piles that do not plug.

- The sum of the individual pile uplift resistance, Q_s, due to shaft resistance, or
- The uplift capacity of the pile group considered as a block as described below

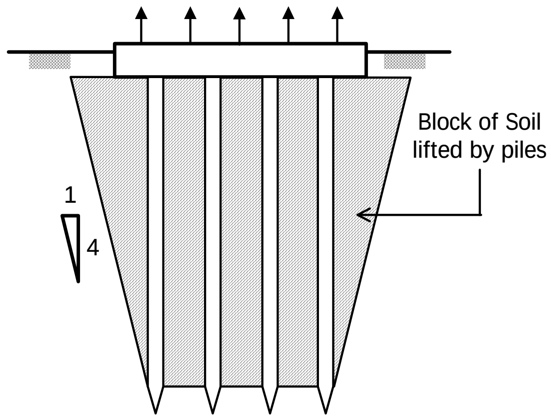


Figure 6-3 Uplift of Group of Closely Spaced Piles in Cohesionless Soils

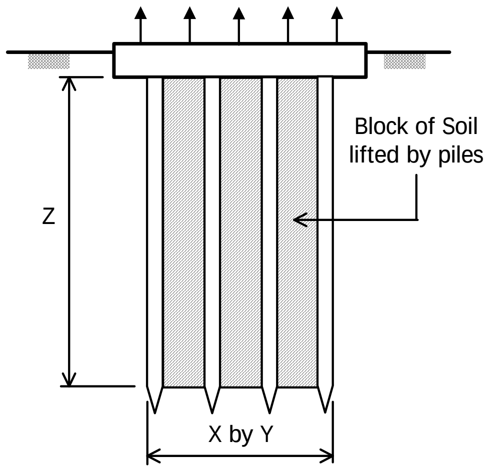


Figure 6-4 Uplift of Group of Piles in Cohesive Soils

Uplift Capacity of Pile Groups in Cohesionless Soil

The weight of the block that will be uplifted, Q_{ug}, shall be determined using a spread of load of 1 in 4 from the base of the pile group taken from Figure 6-3. Buoyant unit weights shall be used for soil below the groundwater level.

Uplift Capacity of Pile Groups in Cohesive Soil

The block used to resist uplift in undrained shear shall be taken from Figure 6-4. The nominal group uplift resistance is taken as:

$$Q_{ug} = (2XZ + 2YZ)\overline{S_u} + W_g$$

where:

X = width of pile group (mm)

Y = length of pile group (mm)

Z = depth of the block of soil below pile cap (mm)

$\overline{S_u}$ = average undrained shear strength along pile shaft (MPa)

W_g = weight of the block of soil, piles and pile cap (N)

6.2.3.3 Lateral Load

Piles are calculated for lateral loads using the following equation called "Displacement Method". This method can be used on the premise that pile cap is rigid.

$$\begin{bmatrix} A_{xx} & A_{xy} & A_{x\alpha} \\ A_{yx} & A_{yy} & A_{y\alpha} \\ A_{\alpha x} & A_{\alpha y} & A_{\alpha\alpha} \end{bmatrix} \begin{Bmatrix} \delta_x \\ \delta_y \\ \delta_\alpha \end{Bmatrix} = \begin{Bmatrix} H_o \\ V_o \\ M_o \end{Bmatrix}$$

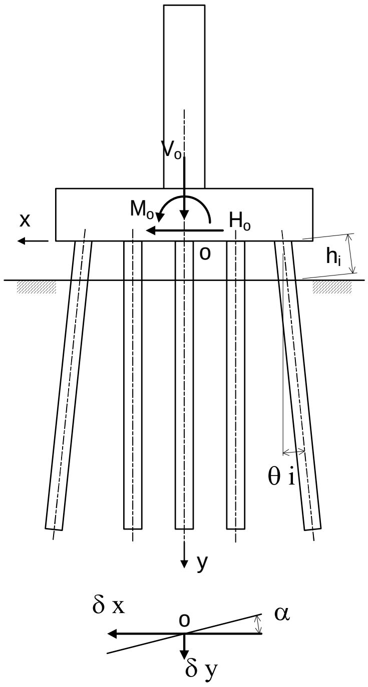


Figure 6-5 Coordinates for Calculation

for which:

$A_{xx} = \Sigma\left(K_1 \cos^2\theta_i + K_v \sin^2\theta_i\right)$

$A_{xy} = A_{yx} = \Sigma\left(K_v - K_1\right)\sin\theta_i \cos\theta_i$

$A_{x\alpha} = A_{\alpha x} = \Sigma\left\{\left(K_v - K_1\right)x_i \sin\theta_i \cos\theta_i - K_2 \cos\theta_i\right\}$

$A_{yy} = \Sigma\left(K_v \cos^2\theta_i + K_1 \sin^2\theta_i\right)$

$A_{y\alpha} = A_{\alpha y} = \Sigma\left\{\left(K_v \cos^2\theta_i + K1 \sin^2\theta_i\right)x_i + K_2 \sin\theta_i\right\}$

$A_{\alpha\alpha} = \Sigma\left\{\left(K_v \cos^2\theta_i + K_1 \sin^2\theta_i\right)x_i^2 + \left(K_2 + K_3\right)x_i \sin\theta_i + K_4\right\}$

where:

H_o = horizontal load acting beneath the footing at the center of pile group (kN)
V_o = vertical load acting beneath the footing at the center of pile group (kN)
M_o = moment acting beneath the footing at the center of pile group (kN.m)
δ_x = horizontal displacement at the center of pile group (m)
δ_y = vertical displacement at the center of pile group (m)
α = angle of rotation at the center of pile group (rad.)
x_i = x coordinate of top of i th pile (m)
θ_i = angle between vertical line and i th pile (rad.)

Table 6-3 Transverse Spring Coefficients of Pile

	$h \neq 0$	$h = 0$	$h \neq 0$	$h = 0$
K_1	$\frac{12EI\beta^3}{(1+\beta h)^3+2}$	$4EI\beta^3$	$\frac{3EI\beta^3}{(1+\beta h)^3+0.5}$	$2EI\beta^3$
K_2, K_3	$K_1\frac{\lambda}{2}$	$2EI\beta^2$	0	0
K_4	$\frac{4EI\beta}{1+\beta h}\frac{(1+\beta h)^3+0.5}{(1+\beta h)^3+2}$	$2EI\beta$	0	0

$Kv = a\frac{A_pE_p}{L}$ (kN/m)

where:

β = characteristic valu of a pile $\beta = \sqrt[4]{\frac{k_H D}{4EI}}$ (1/m)

λ = $h + \frac{1}{\beta}$

k_H = coefficient of lateral ground spring (kN/m^3)

D = pile diameter (m)

EI = flexure rigidity of a pile (kN.m^2)

h = axial length of a pile above the ground level (m); if h < 0, h = 0

a = 0.014(L/D) + 0.72 for driven pile by percussion

0.017(L/D) - 0.014 for driven pile by vibro hammer

0.031(L/D) – 0.15 for cast-in-place conrete pile

A_p = net area of a pile (mm^2)

E_p = modulus of elasticity of a pile (kN/mm^2)

L = pile length (m)

D = pile diameter (m)

To confirm the rigidity of pile cap, thickness of pile cap "h_1" shall satisfy the following equation.

$$\beta_1 \lambda \leq 1.0$$

where;

$$\beta_1 = \sqrt[4]{\frac{3k}{Eh_1^3}} \quad \text{(1/m)}$$

k = k_v vertical ground spring for spread footing (kN/m^3)

k_p converted vertical ground spring for pile foundation (kN/m^3)

$$k_p = K_v \frac{nm}{WB}$$

where:

K_v = axial spring coefficient of a pile (kN/m)

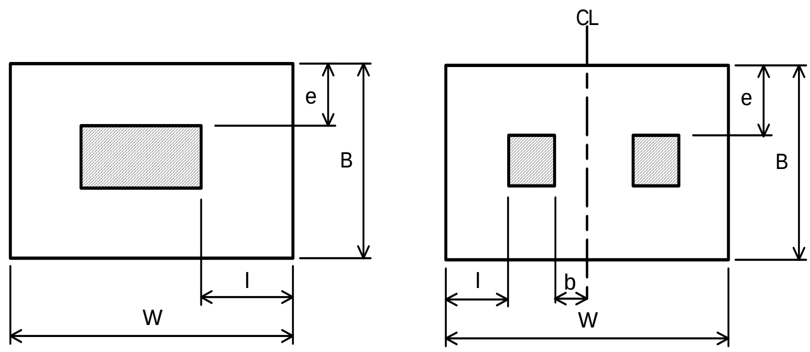
W = length of pile cap (m)

B = width of pile cap (m)

E = modulus of elasticity of pile cap (kN/mm^2)

h_1 = thickness of pile cap (m)

λ = as shown below



Single Footing Continuous Footing

For single footing

λ = max (l,e)

where:

l = W/2 in case l $\geq$ W/2

e = B/2 in case l $\geq$ B/2

For continuous footing

$$\lambda = \frac{\alpha\left(\lambda'^2 + e^2\right)}{\lambda' + e}$$

where:

λ = max (l,b)

α = 1.3

6.2.3.4 Group Axial Load Resistance

The factored bearing resistance of piles, Q_R, is taken as:

$$Q_R = \varphi Q_n = \varphi_g Q_g$$

where:

Q_g = nominal resistance of the group (N)

φ_γ = group resistance factor as specified in Table 6-1

Cohesive Soil

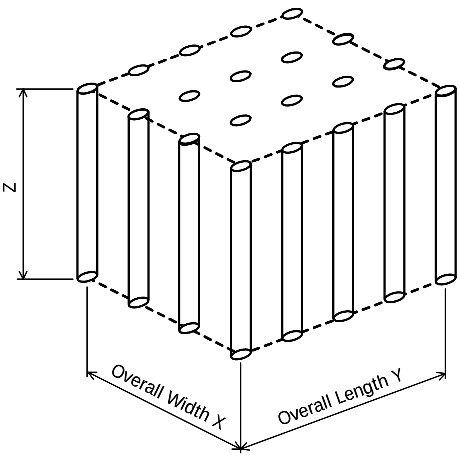
The group resistance shall be the lesser of:

- The sum of the modified individual resistance of each pile in the group with single pile resistance factor by an efficiency factor η, taken as:

Pile Cap / Soil Condition	Center-to-center pile spacings	η
Cap is in firm contact with the ground	At least 2.5 pile diameters / 750mm	1.00
Cap is not in firm contact with the ground and soil is stiff	At least 2.5 pile diameters / 750mm	1.00
Cap is not in firm contact with the ground, and soil at the surface is soft	2.5 pile diameters	0.65
	Intermediate spacings	Linear interpolation
	6.0 pile diameters	1.00

- The resistance of an equivalent pier consisting of the piles and the block of soil within the area bounded by the piles with resistance factor for block failure.

$$Q_g = (2X + 2Y)Z\overline{S_u} + XYN_cS_u$$



for which:

$$N_c = 5\left(1 + \frac{0.2X}{Y}\right)\left(1 + \frac{0.2Z}{X}\right) \quad \text{for} \quad \frac{Z}{X} \le 2.5$$

$$N_c = 7.5\left(1 + \frac{0.2X}{Y}\right) \quad \text{for} \quad \frac{Z}{X} > 2.5$$

$\overline{S_u}$ = average undrained shear strength along the depth of pile penetration (MPa)

S_u = undrained shear strength at the base of the group (MPa)

Cohesionless Soil

The group resistance shall be the sum of the resistance of all the piles in the group with single pile resistance factor and efficiency factor η of 1.00.

6.2.3.5 Group Lateral Load Resistance

Pile group factored resistance for lateral loads shall be taken as:

$$Q_R = \varphi Q_n = \eta \varphi_L \Sigma Q_L$$

where:

Q_L = nominal lateral resistance of a single pile (N)

φ_L = pile group resistance factor as specified in Table 10.5.5-2 of AASHTO LRFD 2007

η = group effective factor; 0.75 for cohesionless soil and 0.85 for cohesive soil

6.2.3.6 Structural Design

Special consideration on buckling shall be given to the piles that extend through water or air, which are assumed to be fixed at some depth below the ground.

6.3 Drilled Shaft

6.3.1 Embedment

Shaft embedment shall be sufficient to provide suitable verifial and latera load capacities and acceptable displacement.

6.3.2 Group Spacing

The center-to-center spacing of drilled shafts shall be equal to or greater than 3.0 diameters, in which case the interaction effects between adjacent shafts shall not be evaluated.

6.3.3 Service Limit state

The provision od section 6.2.3 shall be apply as applicable.

6.3.4 Resistance at Strength Limit State

The resistances that shall be considered include bearing resistance of piles (shaft resistance and tip resistance), uplift resistance of piles, punching of piles through strong soil into a weaker layers and structural resistance of the piles.

6.3.4.1 Bearing Resistance of Single Pile

The equation to calculate factored bearing resistance of pilles is the same as that of driven piles. The resistance factors are given in Table 6-4.

Table 6-4

Resistance Factors for Geotechnical Strength Limit State in Axially Loaded Drilled Shafts

Method / Soil / Condition			Resistance Factor
Ultimate Bearing Resistance of Single-Drilled Shafts	Side Resistance in Clay	α – method (Reese & O'neill 1988)	0.65
	Base Resistance in Clay	Total Stress (Reese & O'neill 1988)	0.55
	Side Resistance in Sand	Touma & Reese (1974) Meyerhof (1976) Quiros & Reese (1977) Reese & Wright (1977) Reese & O'neill (1988)	See Discussion in Article 10.8.3.4
	Base Resistance in Sand	Touma & Reese (1974) Meyerhof (1976) Quiros & Reese (1977) Reese & Wright (1977) Reese & O'neill (1988)	See Discussion in Article 10.8.3.4
	Side Resistance in Rock	Carter & Kulhawy (1988) Horvath & Kenney (1979)	0.55 0.65
	Base Resistance in Rock	Canadian Geotechnical Society (1985) Pressure Method (Canadian Geotechnical Society 1985)	0.50 0.50
	Side Resistance and End Bearing	Load Test	0.80
Block Failure	Clay		0.65
Uplift Resistance of Single-Drilled Shafts	Clay	α – method (Reese & O'neill 1988) β – method (Reese & O'neill 1988)	0.55 0.50
	Sand	Touma & Reese (1974) Meyerhof (1976) Quiros & Reese (1977) Reese & Wright (1977) Reese & O'neill (1988)	See Discussion in Article 10.8.3.7
	Rock	Carter & Kulhawy (1988) Horvath & Kenney (1979)	0.45 0.55
		Load Test	0.80
Group Uplift Resistance		Sand Clay	0.55 0.55

1) Semiempirical Estimates of Drilled Shaft Resistance in Cohesive Soils

Shaft Resistance by Total Stress (α – Method)

The nominal unit side resistance for shafts in cohesive soil loaded under undrained loading conditions is taken as:

$$q_s = \alpha S_u \quad \text{(MPa)}$$

where:

S_u = mean undrained shear strength (MPa)

α = adhesion factor applied to taken for shafts excavated dry in open or cased holes in insentsitive clays as:

S_u (MPa)	< 0.20	0.20 – 0.30	0.30 – 0.40	0.40 – 0.50	0.50 – 0.60	0.60 – 0.70	0.70 – 0.80	0.80 – 0.90	> 0.90
α	0.55	0.49	0.42	0.38	0.35	0.33	0.32	0.31	Treat as Rock

However, the following portion of a drilled shaft as shown in Figure 6-6 shall not be taken to contribute to the development of resistance through skin friction:

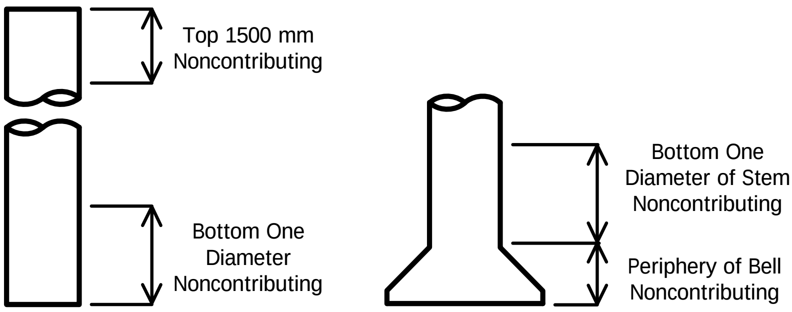


Figure 6-6 Noncontributing Portion of Drilled Shaft to Skin Friction

Tip Resistance

The nominal unit tip resistance is taken as:

$$q_p = N_c S_u \leq 4.0 \quad \text{(MPa)}$$

for which:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9 \qquad S_u \geq 0.024 \quad \text{(MPa)}$$

$N_c = 2\left[1+0.2\left(\frac{Z}{D}\right)\right] \leq 9$ $\qquad S_u < 0.024$ (MPa)

where:

D = diameter of drilled shaft (mm)

Z = penetration of shaft (mm)

S_u = undrained shear strength (MPa), to be obtained within a depth of 2.0 diameters below the tip of the shaft

For drilled shafts in clays with S_u > 0.096 MPa with D > 1900 mm, and for which shaft settlements will not be evaluated, the value of q_p shall be reduced to q_{pr} as follows:

$$q_{pr} = q_p F_r$$

for which:

$$F_r = \frac{760}{12.0aD_p + 760b} \leq 1.0$$

$$a = 0.0071 + 0.0021\frac{Z}{D_p} \leq 0.015$$

$$b = 1.45\sqrt{2.0S_u} \qquad \text{with } 0.50 \leq b \leq 1.50$$

where:

D_p = tip diameter (mm)

2) Estimation of Drilled Shaft Resistance in Cohesionless Soils

<u>Shaft Resistance</u>

The nominal resistance of drilled shafts in sand is determined using any of the five methods specified in Table 6-5.

Table 6-5 Summary of Procedures for Estimating Side Resistance, q_s (MPa) in Sand

Reference	Description
Touma and Reese (1974)	$q_s = K\sigma_v' \tan\varphi_f$ < 0.24 MPa for which: K = 0.70 for $D_b \leq 7500$ (mm) K = 0.60 for $7500 < D_b \leq 12500$ (mm) K = 0.50 for $D_b > 12500$ (mm)
Meyerhof (1976)	$q_s = 0.00096$ N
Quiros and Reese (1977)	$q_s = 0.0025$ N < 0.19 MPa

Reese and Wright (1977)	for $N \le 53$: $q_s = 0.0028\ N$ for $53 < N \le 100$: $q_s = 0.00021\ (N - 53) + 0.15$
Reese and O'Neill (1988)	$q_s = \beta\ \sigma_v' < 0.19$ MPa $(0.25 \le \beta \le 1.20)$ for which: $\beta = 1.5 - 7.7 \times 10^{-3} \sqrt{z}$

where:

N = uncorrected SPT blow count (Blows / 300mm)

σ_v' = vertical effective stress (MPa)

φ_f = friction angle of sand (deg)

K = load transfer factor (DIM)

D_b = embedment of drilled shaft in sand bearing layer (mm)

β = load transfer coefficient (DIM)

z = depth below ground (mm)

Tip Resistance

The nominal tip resistance is calculated using the procedures specified in Table 6-6.

Table 6-6 Summary of Procedures for Estimating Side Resistance, q_s (MPa) in Sand

Reference	Description
Touma and Reese (1974)	Loose $q_p = 0.0$ (MPa) Medium Dense $q_p = 1.5 / k$ (MPa) Very Dense $q_p = 3.8 / k$ (MPa) K = 1.0 for $D_b < 500$ (mm) K = 0.6 for $D_b \ge 500$ (mm) Applicable only if $D_b > 10$ D
Meyerhof (1976)	$q_p = \dfrac{0.013 N_{corr} D_b}{D_p} < 0.13 N_{corr}$ (MPa) for sand $< 0.096 N_{corr}$ (MPa) for nonplastic silts
Reese and Wright (1977)	$q_p = 0.064$ N (MPa) for $N \le 60$ $q_p = 3.8$ (MPa) for $N > 60$
Reese and O'Neill (1988)	$q_p = 0.057$ N (MPa) for $N \le 75$ $q_p = 3.3$ (MPa) for $N > 75$

where:

N = SPT blow count corrected for overburden pressure (Blows / 300mm)

$$= \left[0.77\log_{10}\left(\frac{1.92}{\sigma_v'}\right)\right]N$$

N = uncorrected SPT blow count (Blows / 300mm)

D = diameter of drilled shaft (mm)

D_p = tip diameter of drilled shaft (mm)

D_b = embedment of drilled shaft in sand bearing layer (mm)

σ_v' = vertical effective stress (MPa)

For tip diameters greater than 1270 mm, q_p should be reduced as follows:

$$q_{pr} = \frac{1270}{D_p} q_p$$

6.3.3.2 Uplift

Uplift shall be considered when the force effects are tensile. When piles are subjected to uplift, they should be investigated for both resistance to pullout and structural ability to resist tension and transmit it to the footing.

Both the uplift resistances of a single straight-sided drilled shaft and groups of drilled shafts are calculated in the same manner as that of driven piles due to shaft resistance. The resistance factors shall be taken as specified in Table 6-4.

In determining the uplift resistance of a belled shaft, the side resistance above the bell is neglected, and it can be assumed that the bell behaves as an anchor. The factored uplift capacity of a belled drilled shaft in a cohesive soil, Q_r, is determined as:

$$Q_R = \varphi Q_n = \varphi Q_{sbell}$$

for which:

$$Q_{sbell} = q_{sbell}\ A_u$$

where:

q_{sbell} = $N_u S_u$ (MPa)

A_u = $\pi\left(D_p^2 - D^2\right)/4$ (mm^2)

N_u = uplift adhesion factor (DIM)

= 0.00 (D_b/D_p = 0.75)

= linear interporation (intermediate range)

= 8.00 (D_b/D_p = 2.50)

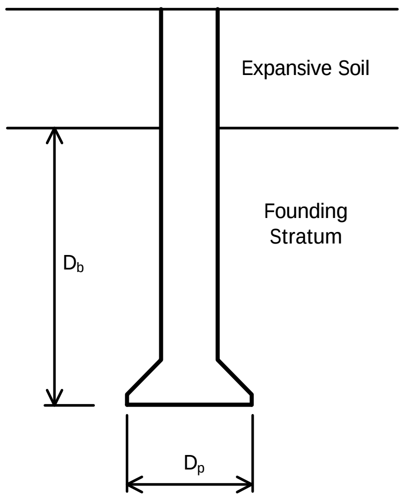


Figure 7-7 Uplift Resistance of Belled Shaft

D_p = diameter of the bell (mm)

D_b = depth of embedment in the founding layer (mm)

D = shaft diameter (mm)

S_u = undrained shear strength averaged over a distance of 2.0 bell diameters ($2D_p$) above the base (MPa)

φ = resistance factors specified in Table 6-4

The top of founding stratum shall be taken at the base of zone of seasonal moisture change.

6.3.3.3 Lateral Load

Lateral load calculations shall be same as those for driven piles.

6.3.3.4 Group Axial Load Resistance

Cohesive Soil

The factored group resistance in cohesive soil is calculated in the same manner as that of driven piles with the resistance factors as specified in Table 6-4.

Cohesionless Soil

The group resistance shall be the sum of the modified resistance of all the piles in the group with single pile resistance factor by efficiency factor η regardless of cap contact with the ground as shown below.

Center-to-center pile spacings	η
2.5 pile diameters	0.65
Intermediate spacings	Linear interpolation
6.0 pile diameters	1.00

6.3.3.5 Group Lateral Load Resistance

Group Lateral load Resistance calculations shall be same as those for driven piles.

6.3.3.6 Structural Design

The structural design of drilled shaft shall be in accodance with the provisions of reinforced concrete.