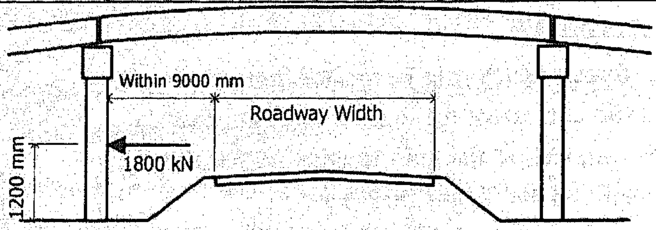


Figure 5 Vehicular Collision Force

(7) ***Train Live Load: TL***

Train Load

A train load of T-26 shall be applied and considered simultaneously with the vehicular design live load. The train load is shown in Figure 6.

Train Load (T-26)

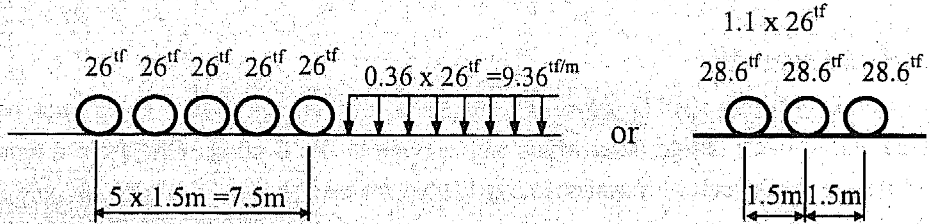


Figure 6 Train Load of T-26

Centrifugal Load

The Centrifugal Load shall act at right angles and horizontally to the truck at the center of gravity of the train load.

Centrifugal Load = Train Load x α (tf)

$$\alpha = \frac{v2}{127R}$$

where:
α = Factor of train load
v = Maximum speed (km/h) of the train traveling on the curved track (m/s) : **to be specified.**
R = Radius of the curve (m)

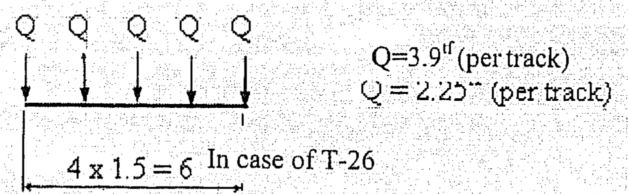
Long Rail Longitudinal Load

Long Rail Longitudinal Load = 1.0tf/m/truck x the overall length of the member (tf/truck)

This value shall not exceed 200tf/truck.

Rolling Stock Lateral Load

The rolling stock lateral load shall act at right angle and horizontally to the rail surface height.



Breaking Load and Starting Load

The breaking load and starting load shall act on the track at the center of gravity height of the train load.

Breaking Load	25% of the characteristic value of train load
Starting Load	25% of the drive wheel axle weight constituting the characteristic value of train load

Note: Loading length of train load shall be within the range of maximum effect on the member

1.3.3. Water Loads: WA

For bridges over water way, the water loads as static pressure, buoyancy and stream pressure shall be adopted with following considerations;

Strength and Service Limit State

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered.

Extreme Event Limit State (with EQ, CT and CV)

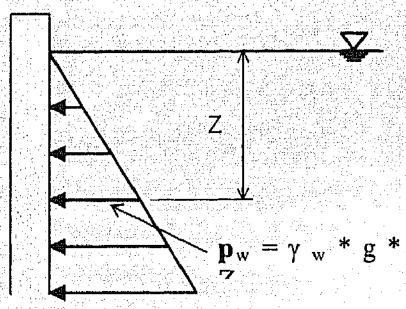
The water loads and scour depths may be based on the mean annual discharge.

Extreme Event Limit State (without EQ, CT and CV)

The structure shall be checked for the consequences of changes in foundation conditions resulting from the check flood for scour.

(1) ***Static Pressure***

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).



Static Water Pressure

(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) ***Stream Pressure***

1) **Longitudinal**

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

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

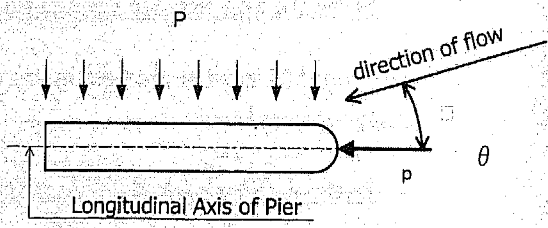
where:
p = pressure of flowing water (MPa)
C_D = drag coefficient for piers as specificed in Table 11
V = velocity of water for the design flood for scour in strength and service limit states and for the check flood for scour in the extreme event limit state (m/s)

Table 11 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 90o 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:



TTransverse Water Pressure

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

where:
p = lateral pressure (MPa)
C_L = lateral drag coefficient specified in Table 12

Table 12 Lateral Drag Coefficient

Angle, θ, between direction of flow and longitudinal axis of the pier	C_L
0°	0.0
5°	0.5
10°	0.7
20°	0.9
>= 30°	1.0

(4) <u>Wave load</u>

Wave load on the bridge structures shall be considered for exposed where the development of significant wave forces may occur.

1.3.4. Wind Load

(1) <u>Horizontal Wind Load</u>

1) General

This Article provides design wind loads for conventional bridge structures. For long span, specific wind climate studies should be carried out to determine the wind effects.

The design wind velocity, V, shall be determined from:

$V = V_B * S$ where:

V_B = basic 3 second gust wind velocity with 100 years return period appropriate to the Wind Zone in which the bridge is located, as specified in Table 13

S = correction factor for upwind terrain and deck height, as specified in Table 14

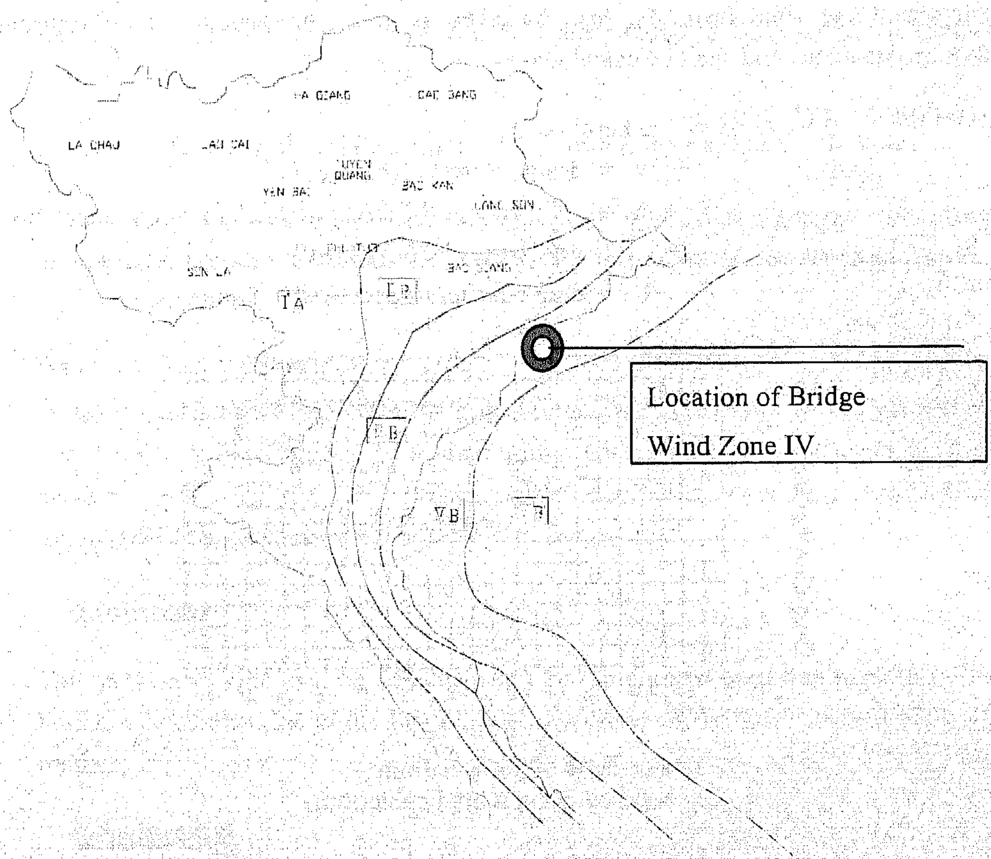


Figure 7 Wind Zone Map in Vietnam

Table 13 Values of V_B for Wind Zones in Vietnam

Wind zone (TCVN2737-1995)	V_B (m/s)
I	38
II	45
III	53
IV	59

Table 14 Values of S

Height of bridge deck above surrounding ground or water level (m)	Open country or open water	Wooded country or built-up areas, with trees or buildings up to a maximum height of about 10 m	Built-up areas with buildings predominantly over 10 m high
10	1.09	1.00	0.81
20	1.14	1.06	0.89
30	1.17	1.10	0.94
40	1.20	1.13	0.98
50	1.21	1.16	1.01

2) Wind Load on Structures: WS

<u>Transverse Wind Load</u>

The transverse wind load, P_D, shall be taken as acting horizontally at the centroids of the appropriate areas, and shall be calculated as:

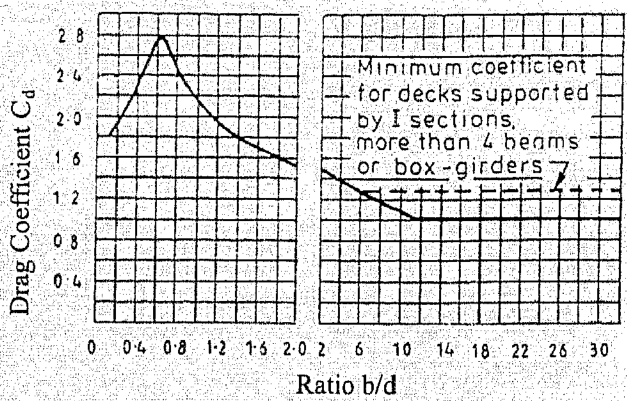
$$P_D = 0.0006V^2 A_t C_d \geq 1.8A_t \quad \text{(kN)}$$

where:

V = design wind velocity (m/s)

A_t = solid area of the structure or element for calculation of transverse wind load (m^2)

C_d = drag coefficient specified in Figure 8



b=overall width of bridge between outer faces of parapets(mm)
d=depth of superstructure including solid parapets if applicable(mm)

Figure 8 Drag Coefficient Cd for Superstructures with Solid Elevation

The area of the structure or element under consideration A_t, shall be the solid area in normal projected elevation, without live load, subjected the following provisions;

For superstructures with solid parapets:	The area of superstructure shall include the area of the solid windward parapet, but the effect of the leeward parapet need not be considered.
For superstructures with open parapets:	The area of superstructure shall include both windward and leeward parapets separately considered. Where there are more than two parapets, only those two having the greatest unshielded effect shall be considered.
For truss girder superstructures:	The wind force shall be calculated for each component separately, both windward and leeward, without considering shielding.
For piers:	Shielding shall not be considered

Longitudinal Wind Load

The longitudinal wind load shall be considered as follows;

For piers, abutments, truss girder superstructures and other superstructure forms which represent a significant surface area to wind loads parallel to longitudinal centerline similar way to those for transverse wind loads

For superstructure with solid elevation: 0.25 times the transverse wind load

Longitudinal and transverse wind loads shall be applied as separate load cases and, where appropriate, the structure shall be checked for the effect of intermediate angles of wind by resolution of forces.

3) **Wind Load on Vehicles: WL**

When considering STRENGTH III load combination, the design wind load shall be applied to both structure and vehicles. And longitudinal and transverse wind loads shall be applied as separate load cases and, where appropriate, the structure shall be checked for the effect of intermediate angles of wind by resolution of forces.

Transverse

The transverse wind load on vehicles shall be represented by a line load of 1.50 kN/m acting horizontally, transverse to the longitudinal centerline of the structure and 1800 mm above the roadway.

Longitudinal

The longitudinal wind load on vehicles shall be represented by a line load of 0.75 kN/m acting horizontally, parallel to the longitudinal centerline of the structure and 1800 mm above the roadway.

(2) Vertical Wind Load

In case the angle of inclination of the wind to the structure less than 5 degrees, a vertical wind load, Pv, shall be taken as acting at the centroid of the appropriate area, and shall be calculated as:

$P_v = 0.00045V^2A_v$ (kN)

where:
V = design wind velocity (m/s)
A_v= plan area of the bridge deck or element for calculation of vertical wind load (m^2)

This load shall be applied only for limit states that do not involve wind on live load, and only when the direction of wind is taken to be perpendicular to the longitudinal axis of the bridge.

1.3.5. Earthquake Effects: EQ

Earthquake loads shall be taken to be horizontal force effects for rigid-frame superstructures, substructures, foundations and connections between superstructures and substructures.

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

These loads are determined based on the following items.

- Acceleration Coefficient (AC) at each bridge
- Importance Categories (IC) for each bridge
- Seismic Zone based on AC for each bridge
- Site Effects (S) based on soil profile type
- Period of Vibration of the m-th mode (Tm) for the structure
- Response Modification Factor (R) for the substructures and connections

(1) Analysis for Earthquake Loads

The minimum analysis requirements for seismic effects shall be as specified in Table 15 depend on structural type, seismic zone, importance category, and part of the structure.

The connections between the superstructure and substructure shall be designed for the minimum force requirements.

Also the minimum seat width requirement shall be satisfied.

- UL = uniform load elastic method
- SM = single-mode elastic method
- MM = multimode elastic method
- TH = time history method

Table 15 Minimum Analysis Requirements for Seismic Effects

Seismic Zone	Single- Span Bridges	Multi-span Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		Regular	Irregular	Regular	Irregular	Regular	Irregular
1	No need	No need	No need	No need	No need	No need	No need
2	No need	SM/UL	SM	SM/UL	MM	MM	MM
3	No need	SM/UL	MM	MM	MM	MM	TH

(2) Acceleration Coefficients (AC) and Seismic Zone

The seismic zone of each bridge shall be determined base on the acceleration coefficient using following table:

Table 16 Seismic Zones

Acceleration coefficients	Seismic zone	MSK – 64 class
A <= 0.09	1	Class <= 6.5
0.09 < A <= 0.19	2	6.5 < Class <= 7.5
0.19 < A < 0.29	3	7.5 < Class <= 8

According to the Vietnamese Design Code TCXDVN 375:2006 "Design of Structures with Seismic Isolation", project site is located on the seismic zone 7, acceleration coefficient A=0.1291.

(3) *Importance Categories*

The Owner shall classify the bridge one of three Importance Categories as follows;

- Critical Bridges
- Essential Bridges
- Other Bridges

(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 are shown in Table 17.

Table 17 Site Coefficients

Site Coefficient	Soil Profile Type			
	I	II	III	IV
S	1.00	1.20	1.50	2.00

Where the soil profiles are follows;

Soil Profile Type I

Rock of any description, either shale - like or crystalline in nature, or Stiff soils where the soil depth is less than 60 m, and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays

Soil Profile Type II

Stiff cohesive or deep cohesionless soils where the soil depth exceeds 60 m and the soil types overlaying the rock are stable deposits of sands, gravels, or stiff clays

Soil Profile Type III

Soft to medium-stiff clays and sands, characterized by 9 m or more of soft to medium-stiff clays

Soil Profile Type IV

Soft clays or silts greater than 12 m in depth

(5) ***Elastic Seismic Response Coefficient***

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

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

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

For soil profiles III and IV, and for modes other than the fundamental mode that have periods less 0.30s, C_{sm} shall be taken as:

- $C_{sm} = A(0.8+4.0\ T_m)$

If the period of vibration for any mode exceeds 4.0s, the value of C_{sm} for that mode shall be taken as;

- $C_{sm} = 3.0\ AS / T_m^{4/3}$

(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.

Table 18 Response Modification Factors-Substructures

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

Table 19 Response Modification Factors-Connections

Connection	All importance 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

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

(7) Combination of Seismic Force Effects

The following two load cases combining elastic member forces resulting from earthquakes to the longitudinal and transverse axes of the bridge, should be considered.

Load Case 1: 1.0 FL+ 0.3 FT

Load Case 2: 0.3 FL+ 1.0 FT

where:

FL= absolute elastic member forces due to an earthquake to the longitudinal axis

FT= absolute elastic member forces due to an earthquake to the transverse axis

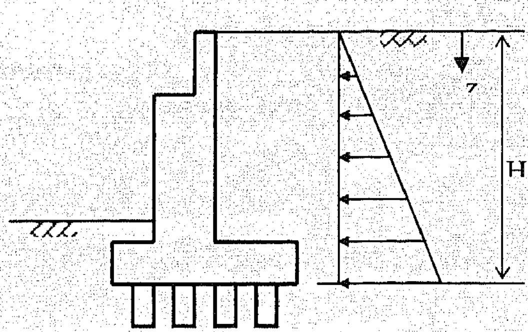
1.3.6. Earth Pressure: EH

Earth pressure shall be considered as a function of the:

- Type and density of soil,
- Water content,
- Soil creep characteristic,
- Degree of compaction,
- Location of groundwater table,
- Earth-structure interaction,
- Amount of surcharge,
- Earthquake effects.

(1) Basic Earth Pressure

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



Earth Pressure

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

where:

p = basic 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 (m/s^2)

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of 0.4H above the base of the wall, where H is the total wall height, measured from the surface of the ground to the bottom of the footing.

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

$Ko = 1 - \sin\varphi_f$
: for normally consolidated soils
$Ko = (1 - \sin\varphi_f)(OCR)^{\sin\varphi f}$
: for overconsolidated soils

where:
φ_f = effective friction angle of soil
Ko = coefficient of earth pressure at rest
OCR = overconsolidation ratio (refer to Table 20)

Table 20 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

(3) Active Lateral Earth Pressure Coefficient, Ka

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

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

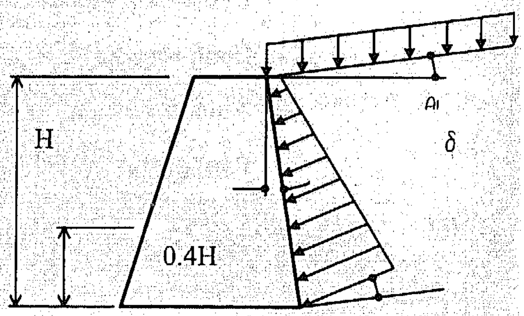
where:

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)



Earth Pressure

(4) Passive Lateral Earth Pressure Coefficient, Kp

For noncohesive soils, values of the coefficient of passive lateral earth pressure Kp may be taken from Figure 1 of Specification 22 TCN 272-05 for the case of a sloping or vertical wall with a horizontal backfill or from 3.11.5.4-2 of Specification 22 TCN 272-05 for the case of a vertical wall and sloping backfill.

For conditions that deviate from those described in Figures 1 and 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.

For cohesive soils, passive pressures may be estimated by:

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

pp = lateral earth pressure (MPa)
γ_s = density of soil (kg/m³)
z = depth below the surface of soil (mm)
c = unit cohesion (MPa)
K_p = coefficient of passive lateral earth pressure

(5) ***Seismic Active Earth Pressure Coefficient Kae:***

Seismic active earth pressure Pae shall be taken as:

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

for which:

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

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

where:

θ_0 = arc tan (kh/(1-kv)) (deg)
kh = horizontal acceleration coefficient
kv = vertical acceleration coefficient

(6) ***4Seismic Passive Earth Pressure Coefficient Kpe:***

Seismic active earth pressure Pae shall be taken as:

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

for which:

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

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

where:

θ_0 = arc tan (kh/(1-kv)) (deg)

kh = horizontal acceleration coefficient

kv = vertical acceleration coefficient

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

(1) *Uniform Temperature: TU*

The maximum and minimum average bridge temperature specified in TVCN are shown in Table 21. The difference between the maximum and minimum average bridge temperature and the base construction temperature assumed in the design shall be used to calculate thermal deformation effects. These are based on shade air temperature ranges of 0 °C to +45 °C north of latitude 16° N (Hai Van Pass) and +5 °C to +45 °C south of latitude 16° N.

The setting temperature of the bridge shall be taken as the actual air temperature averaged over the 24-hour period immediately preceding the setting event.

These temperatures should be reviewed in considering with the meteorological data of the site.

Table 21 Bridge Temperature Ranges

Climate Zone	Concrete superstructure	Concrete deck on steel girders or box	Steel deck on steel girders or box
North of Latitude 16 Deg. N (Hai Van Pass)*	+5 °C to +47 °C	+1 °C to +55 °C	-3 °C to +63 °C
South of Latitude 16 Deg. N (Hai Van Pass)	+10 °C to +47 °C	+6 °C to +55 °C	+2 °C to +63 °C

*: For sites north of latitude 16° N and at an elevation above sea level greater than 700 m, the minimum temperature in the Figure 9 shall be reduced by 5 °C.

(2) *Temperature Gradient: TG*

The effect of vertical differential temperature gradients through a bridge superstructure shall be derived for both positive temperature differential conditions (top surface hotter) and negative temperature differentials (top surface cooler). Dimension "A" in Figure 9 shall be taken as;

For concrete superstructures that are 400 mm or more in depth ----- A = 300 mm:

For concrete sections shallower than 400 mm ----- A = 100 mm less than actual depth

For steel/concrete composite superstructures ----- t = depth of concrete deck

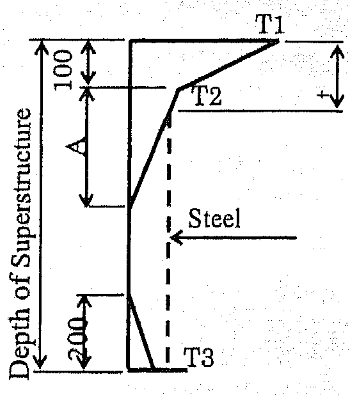


Figure 9 Vertical Temperature Gradient

The temperature gradients given in Table 22 apply to bridge decks with 100 mm thickness of surfacing. Where a different surfacing thickness is used, the values should be adjusted accordingly.

Table 22 Temperature Gradient (°C)

	T1	T2	T3
Positive	+23	+6	+3
Negative	-7	-1	0

(3) ***Creep and Drying Shrinkage***

Influences of creep and drying shrinkage are varied from condition of the land, quality of the materials, dimension of the members, the ages, and the erection methods. To evaluate the differences of the ages varied from the erection methods accurately have to consider not only the amount of the final creep drying shrinkage but also the erection order and progress of the creep drying shrinkage. In this project, the regulations for creep drying shrinkage of CEB-FIP model code (MC78) mentioned in AASHTO 5.4.2.3.1 is applied.

1) Creep Coefficient

According to MC78, the total strain related the stress at t when the fixed continue stress is acted by t0 is shown in the following equation as the sum of elastic strain occurred at t0 and creep strain occurred from between t_0 and t.

$$\Sigma\ \varepsilon\,(t, t_0) = (\sigma_0)/Ec(t_0) + (\sigma_0)/Ec{,}28 * \phi\,(t, t_0)$$

Where, $Ec(t_0)$: Elastic modulus for concrete of the age t_0
$Ec{,}28$: Elastic modulus for concrete of the age for 28days.
$\varphi(t, t_0)$: Creep strain progress between from t0 to t given in MC78.

According to the equation, elastic modulus for concrete decided the creep strain by MC78 have to use the elastic modulus for concrete of the age for 28days all the time.

The creep coefficient is decided by the following equation:

$$\phi\, i = \phi\, d_0 * \beta_d\,(t'i - t'i\text{-}1) + \phi\, f_0\, \{\beta f(t'i) - \beta f(t'i\text{-}1)\}$$

where,	
$\Delta\varphi i$:	Creep coefficient progress between ti-1 and ti.
φd_0:	Basic creep coefficient 0.4 against late elastic strain.
φf_0:	Using the number of Table 23 according to the environmental condition by basic creep coefficient against the flow.
$\beta_d(t'i - t'i\text{-}1)$:	Using the number of the Table 24 by the coefficient for the changes with times of late elastic strain.
$\beta f\,(t'i)$:	Using the number of Table 24 according to the theoretical thickness of materials by the coefficient for the changes with times of flow.
t i :	The ages of concrete after loading.
t'i :	Effective age fixed by the kinds of cement and surround average temperature during concrete hardening. $=\alpha/30\Sigma ti\,(T\,(ti) + 10) * ti$
α :	Using the coefficient related the hardening speed of concrete. normal concrete: 1.0, high speed concrete: 2.0
T(t i) :	Average temperature of t i days.
hth :	$\lambda * Ao / u$ (theoretical thickness)
λ :	Using the number of Table 23 by coefficient related the environmental condition.

A_0 : Sectional area of the materials.
u : Circumference length(m) of the connected section to the air of material section.

Table 23 φf_0, εs_0 and λ

Environment Condition		φf_0	εs_0 (×10−6)	λ
Relative Humidity (%)	100 (Underwater)	0.8	−100	60
	90	1.3	100	10
	70	2.0	250	3
	40	3.0	400	2

Table 24 Coefficient βd(ti) and Coefficient βf(t'i)

Effective Age (Day)	βd(t'i-t'-1)	βf(t'i) against hth (cm)					
		<=5	10	20	40	80	>=160
1	0.280	-	-	-	-	-	-
2	0.300	-	-	-	-	-	-
3	-	0.240	0.210	0.190	0.170	0.155	0.140
5	0.350	0.345	0.310	0.270	0.235	0.210	0.185
10	0.400	0.505	0.440	0.380	0.328	0.280	0.235
20	0.465	0.685	0.575	0.500	0.420	0.350	0.280
30	0.580	—	—	—	—	—	—
50	—	0.964	0.810	0.690	0.562	0.443	0.330
100	0.700	1.195	1.025	0.850	0.680	0.52	0.375
200	0.830	1.395	1.215	1.020	0.800	0.603	0.435
500	0.945	1.600	1.413	1.208	0.980	0.750	0.566
1000	0.985	1.698	1.514	1.320	1.107	0.884	0.703
2000	1.000	1.762	1.589	1.416	1.217	1.010	0.842
5000	1.000	1.820	1.660	1.510	1.330	1.148	1.000
10000	1.000	1.846	1.695	1.545	1.383	1.225	1.085
20000	1.000	1.850	1.700	1.500	1.400	1.250	1.120
∞	1.000	1.850	1.700	1.550	1.400	1.250	1.120

2) Drying Shrinkage

Drying shrinkage is decided by the following equation,

$\varepsilon cs\ i = \varepsilon so\ (\beta s\ (t''i) - \beta s\ (t''i - 1))$

where,

$\Delta\varepsilon cs,i$: Drying shrinkage which progress during ti – 1 to ti

εso :Using the number of Table 23 according to the basic drying shrinkage strain and environmental condition.

$\beta s(t''i)$:Using the number of Table 25 according to the theoretical thickness of materials by the coefficient for the changes with times of drying shrinkage.

t''i :Effective age fixed by the surrounding average temperature during concrete hardening.

$=1/30\Sigma ti\ (T\ (ti) + 10)*ti$

Table 25 Coefficient βs(t''i)

Effective Age (Day)	Coefficient βs(t''i) against hth (cm)					
	<=5	10	20	40	80	>=160
1	0.110	0.040	0.010	0.0	0.0	0.0
2	0.170	0.080	0.020	0.0	0.0	0.0
5	0.290	0.160	0.055	0.005	0.005	0.0
10	0.420	0.240	0.100	0.005	0.020	0.0
20	0.560	0.340	0.160	0.060	0.030	0.0
50	0.760	0.510	0.270	0.120	0.055	0.010
100	0.900	0.650	0.375	0.185	0.085	0.020
200	1.020	0.780	0.490	0.260	0.120	0.045
500	1.100	0.910	0.660	0.410	0.210	0.090
1000	1.160	0.980	0.770	0.550	0.340	0.175
2000	1.190	1.040	0.840	0.660	0.500	0.310
5000	1.200	1.050	0.885	0.750	0.660	0.510
10000	1.200	1.050	0.895	0.790	0.725	0.640
20000	1.200	1.050	0.900	0.800	0.750	0.700
∞	1.200	1.050	0.900	0.800	0.750	0.700

(4) Friction Forces: FR

Forces due to friction on the sliding or rotating surface shall be considered. The value of friction coefficients are depend on the specification of each product. Based on the former projects in Vietnam, the friction coefficients of elastmetric bearing shall be as 0.15.

(5) Vessel Collision: CV

All bridges crossing navigable waterways shall be designed for vessel collision with the substructure and, where appropriate, the superstructure.

The Owner shall establish and/or approve the design vessel(s), design velocity, and any specific requirements for the bridge in consideration with the Vietnam Inland Waterways Bureau or the Vietnam Marine Authority, as appropriate.

1) Design Vessel

The design vessels and their dimensions are given for various classes of navigable waterway shown in Table 26 and Table 27.

Table 26 Design Vessels for Classes of Navigable Waterway

Class of navigable waterway	Design vessel tonnage (dwt)	
	Self-propelled vessel	Towed barge
I	2000	500
II	1000	500
III	300	400
IV	200	400
V	100	100
VI	40	100