WELCOME

30-DEC-2022

Session 4

Maldives Link Project

By

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Tech TANGENT Solutions Pvt Ltd

Content

- Introduction to Project
- Loadings as per Eurocode
- Design of Monopile

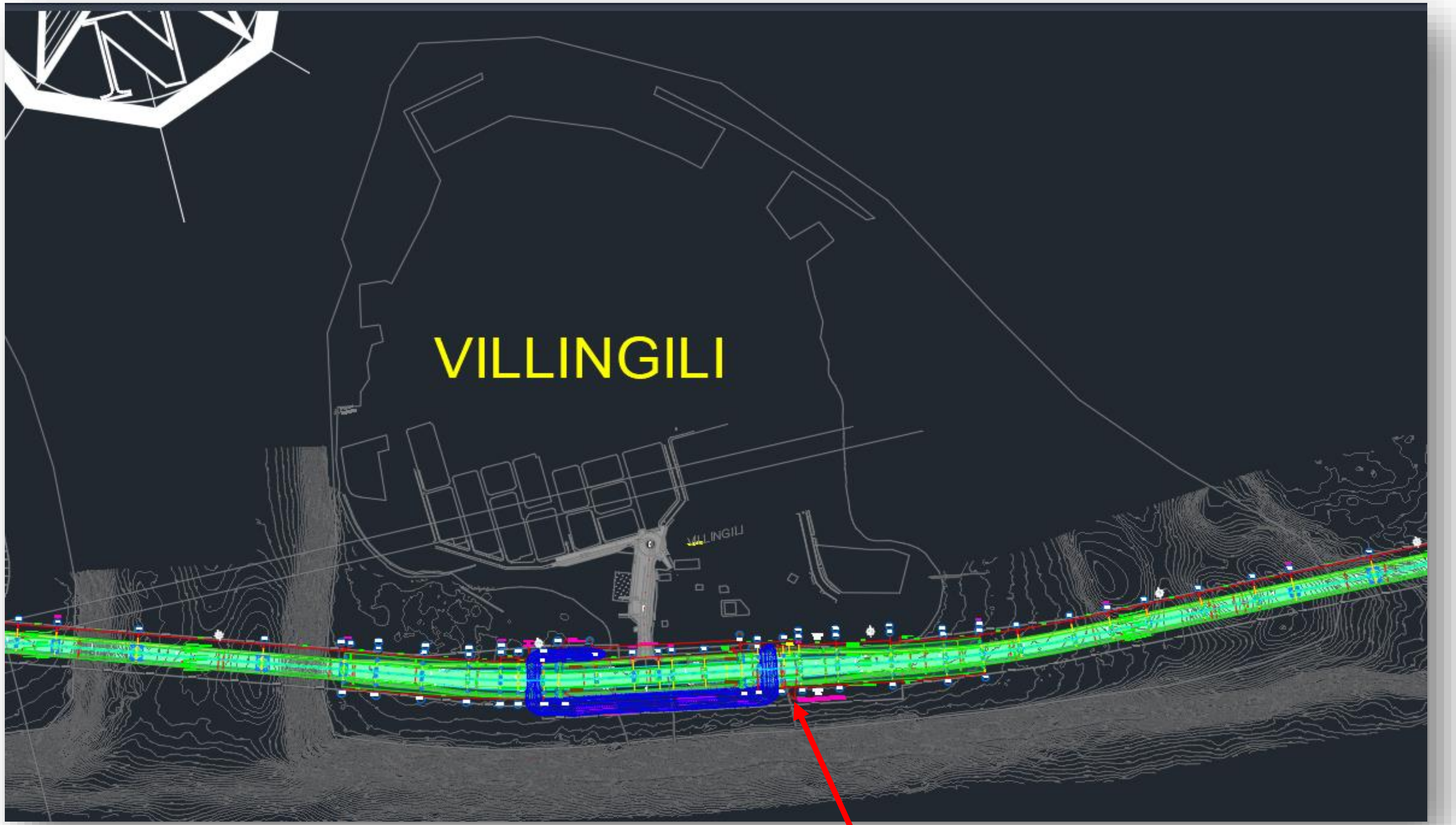
OVERVIEW OF THE PROJECT

- Project Name: **Greater Male Connectivity- Male to Thilafushi Link Project**
- Project Length: **6.73 km**
- Construction Cost: **4400 cr**
- Completion Period: **32 months**
- Owner: **Ministry of National Planning, Housing and Infrastructure, Government of Maldives**
- Contractor: **AFCONS Infrastructure Ltd**
- Design Consultant: **AECOM and Tech Tangent Solutions Pvt. Ltd.**
- Independent Engineer : **SYSTRA**

Location Details

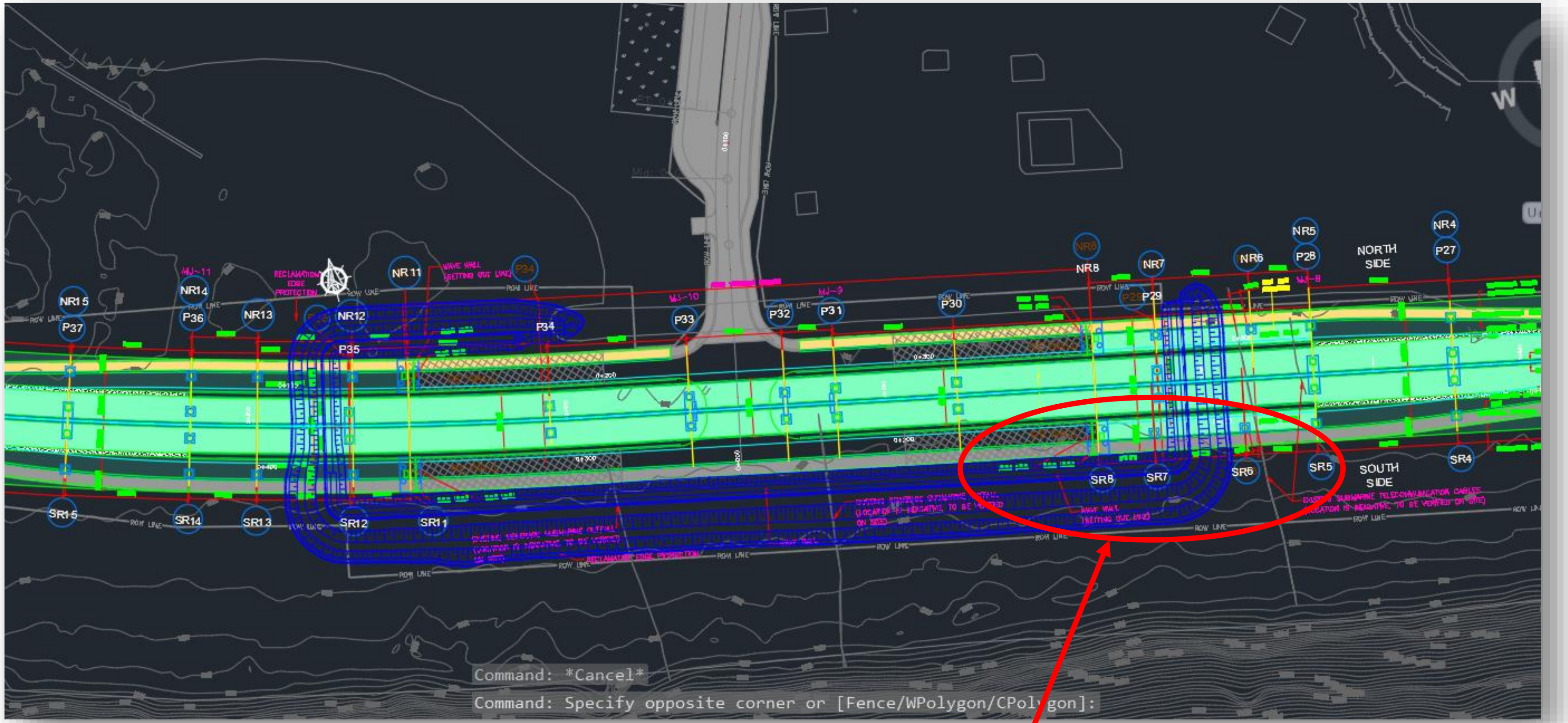
Module to be designed

Location Details



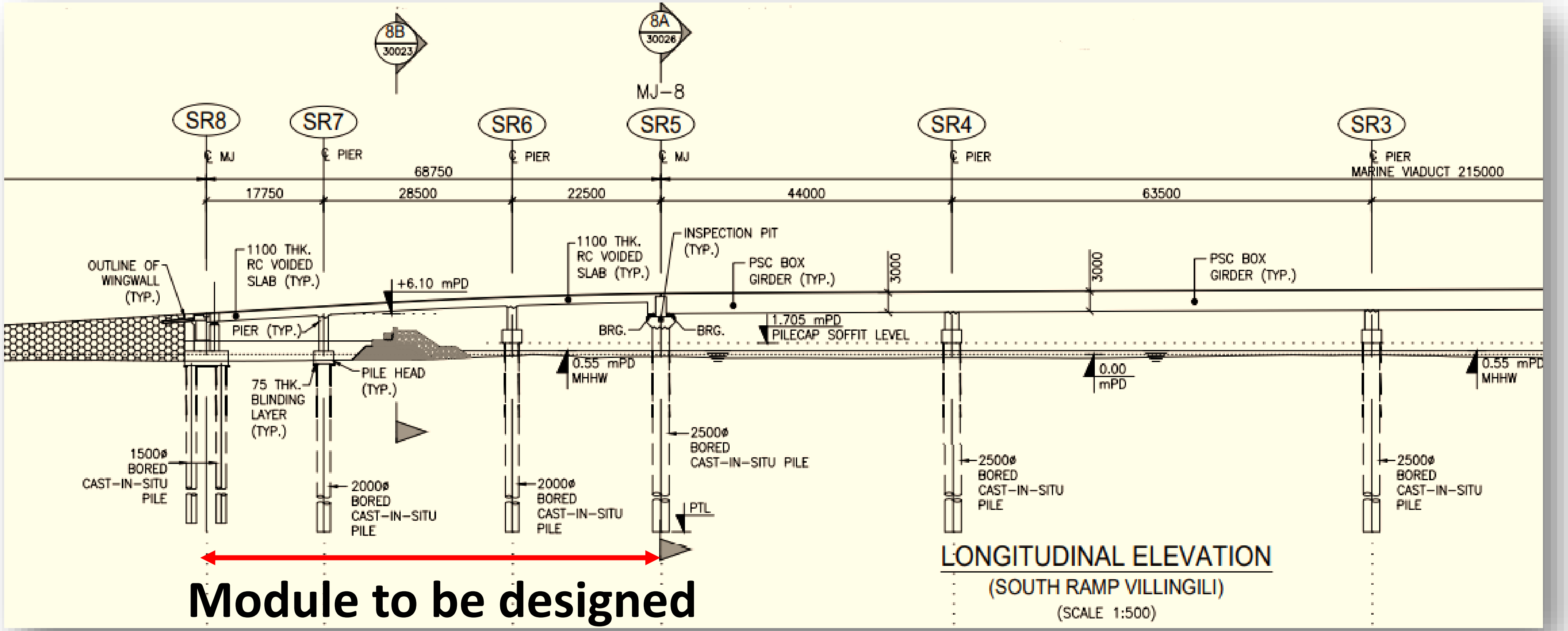
Module to be designed

Location Details



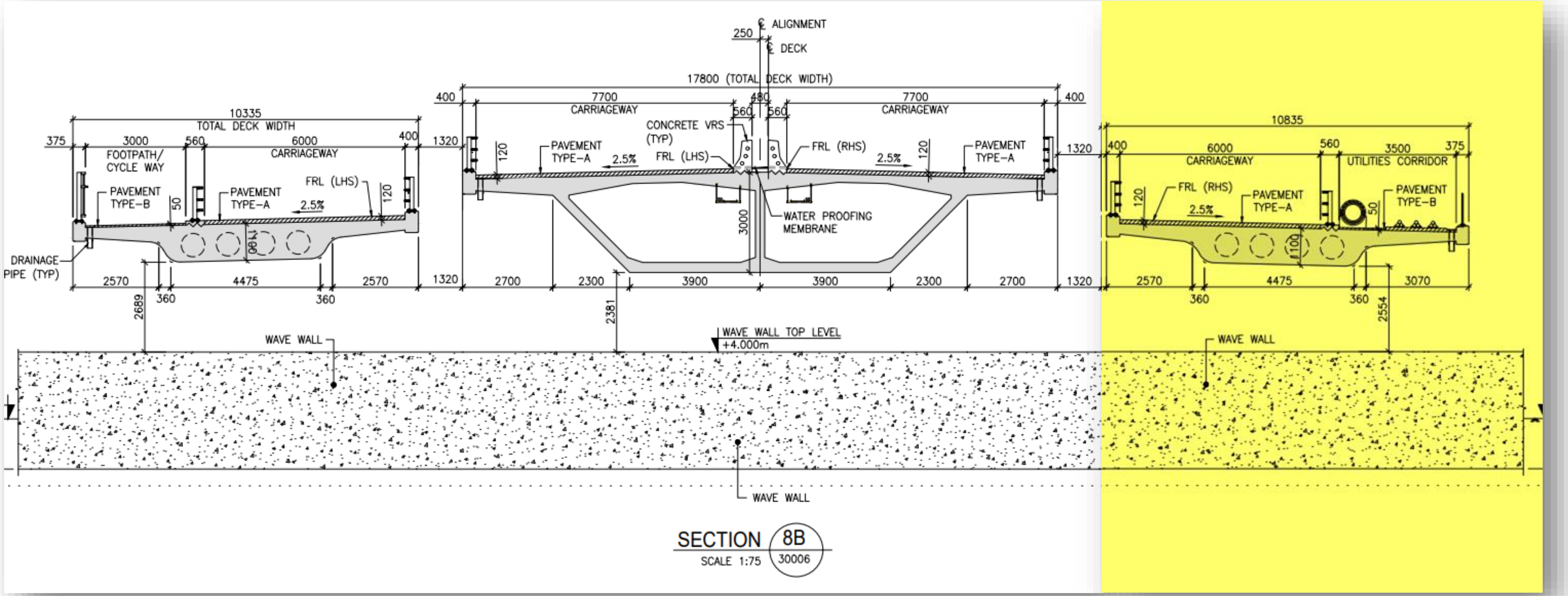
Module to be designed

Longitudinal Elevation



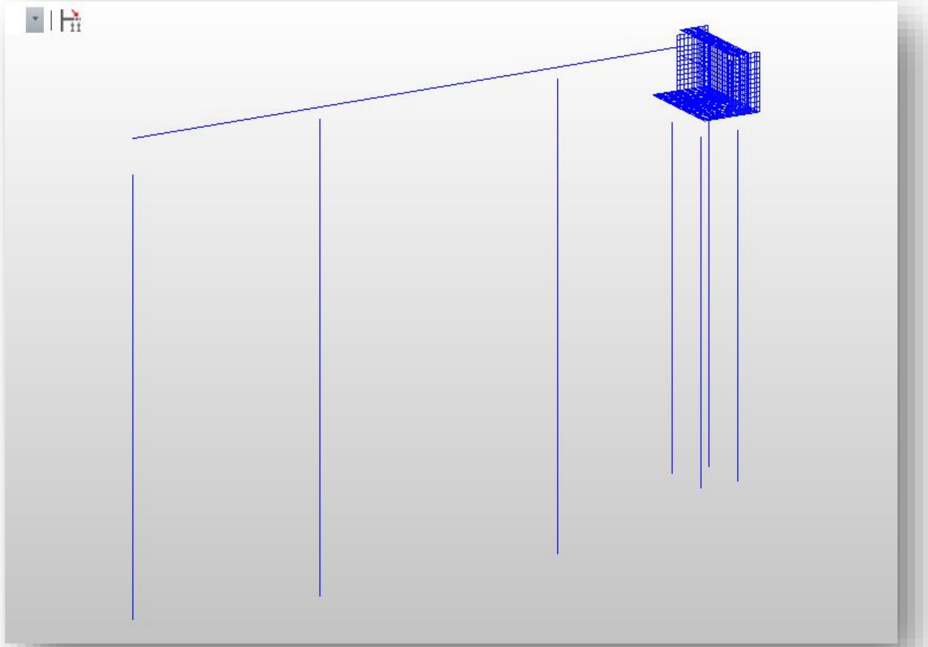
1. Superstructure Monolithic with Intermediate Two Piers.
2. Bearings are provided at end pier and Abutment location.
3. 2.5m dia Monopile at Pier Location, 4 Nos-1.5m Piles at Abutment location

Cross Section

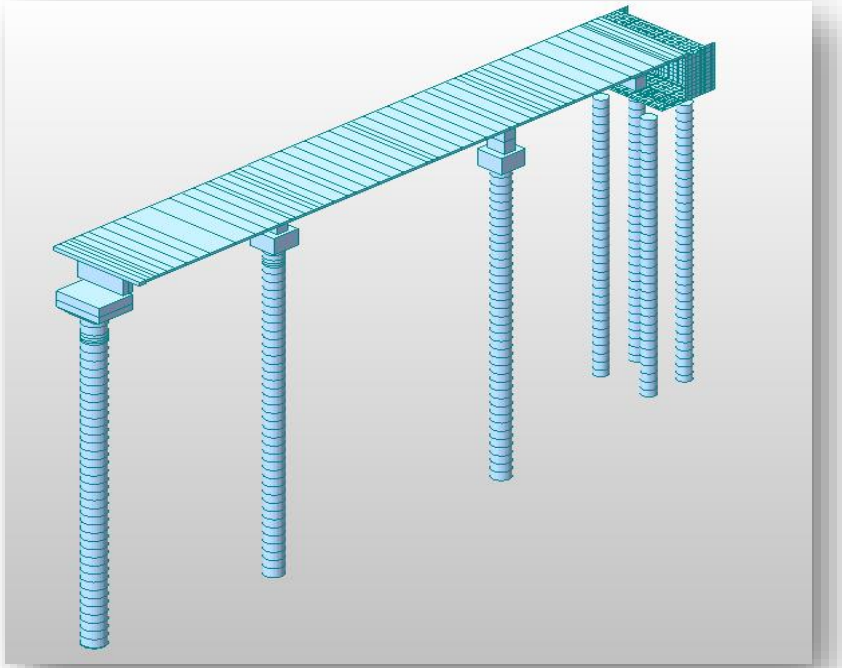


1. Precast Segmental Post-Tensioned Box Girder with continuous superstructure.
2. RCC Voided Deck Slab (To be designed)

Span Arrangement – Midas Model



Line Model (Midas)



Cross sectional View (Midas)

Material Characteristics and Cover Requirements

- **Material Characteristics**

1. Pile, Pile cap, Pier/Abutment **- C40**
2. Superstructure - **C45**
3. Reinforcement - **Fe550D**

- **Cover Requirements**

1. Pile, Pile cap, Pier/Abutment - **75mm**
2. For Piles in Splash zone - **99mm**
3. Superstructure - **55mm**

LOADS

1. DL
2. SIDL
3. Live Loads
4. Seismic
5. Temperature
6. Wind
7. Tsunami Loads
8. Water Current
9. Ship Impact

Eurocode

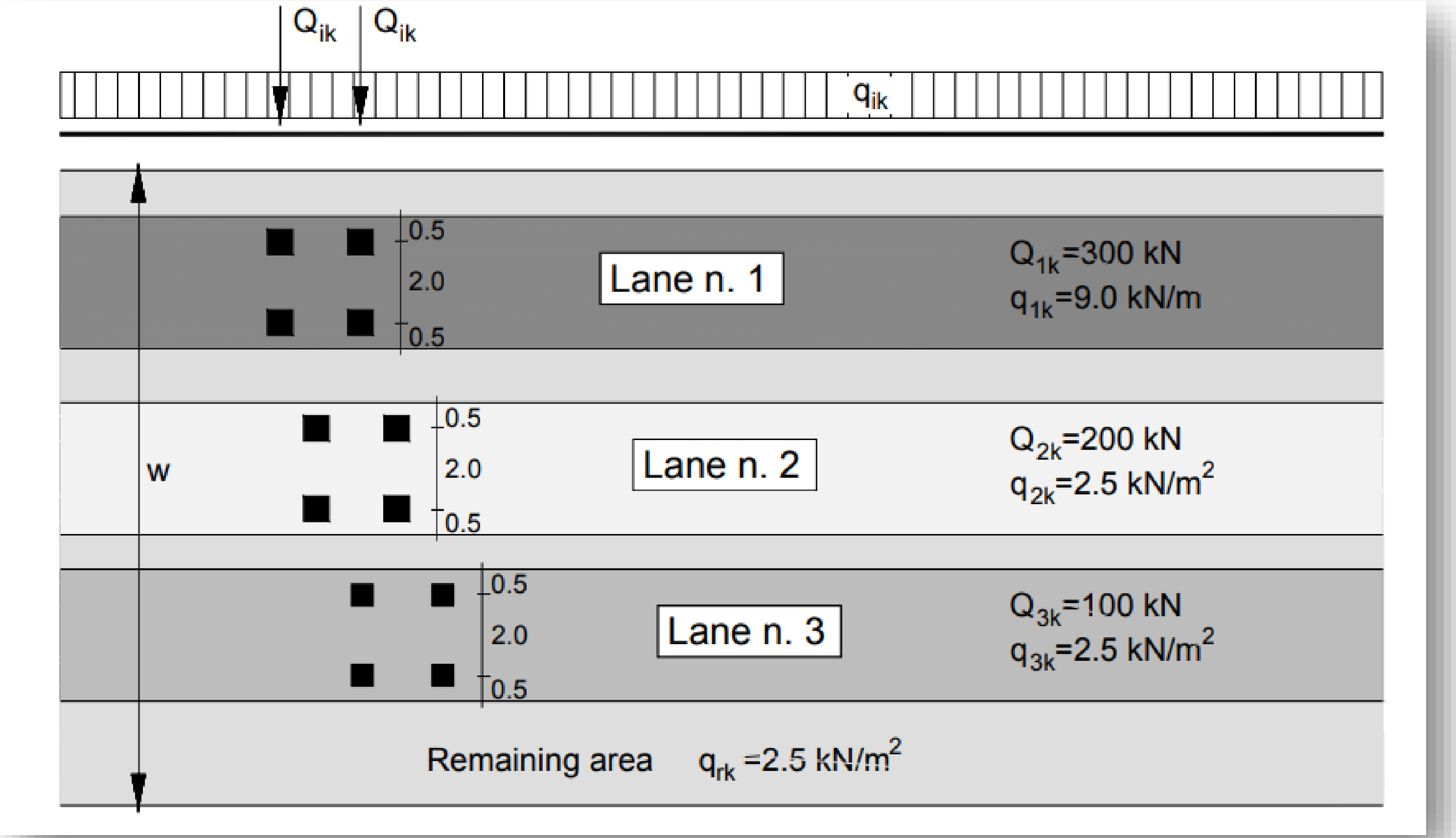
- EN 1991-2-2003 (For LL)
- EN 1991-1-4-2005 (For wind)
- EN 1991-1-5-2003 (For Temp.)
- EN 1992-1-1-2004 (For Pile/Pier design)
- ASCE 7-16 Supplements-(For Tsunami)

Live Loads

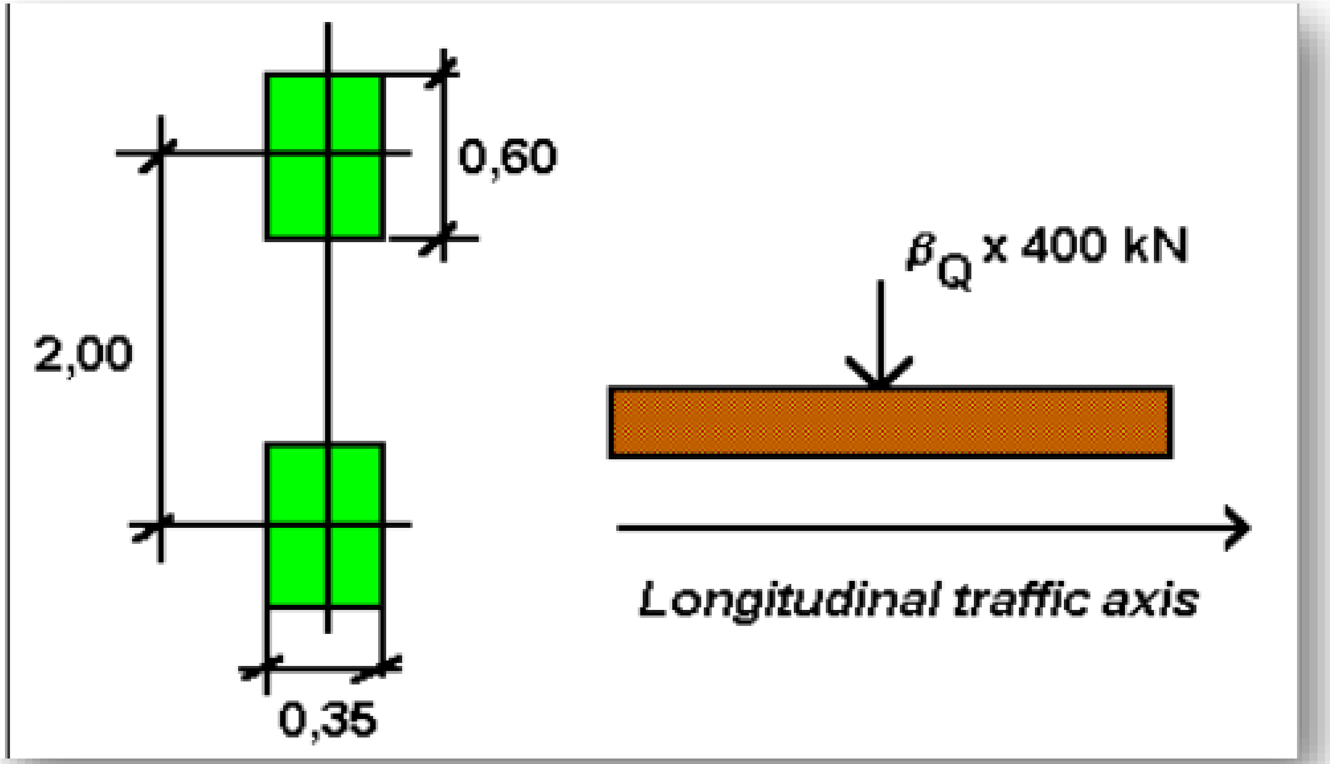
BS EN 1991-2:2003
EN 1991-2:2003 (E)

- Load Model Nr. 1 - Concentrated and distributed loads (main model)
- Load Model Nr. 2 - Single axle load
- Load Model Nr. 3 - Set of special vehicles *(Can be specified by NA)*
- Load Model Nr. 4 - Crowd loading : 5 kN/m^2

1. Load Model 1

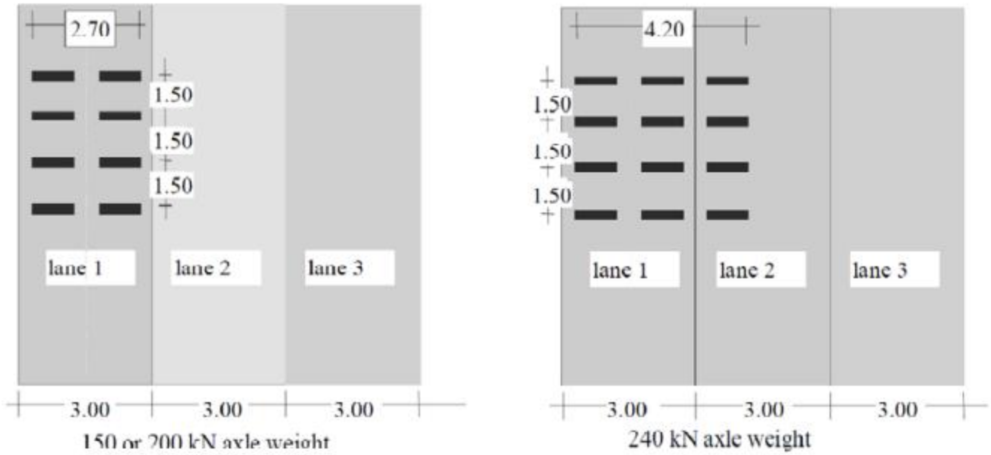


2. Load Model 2

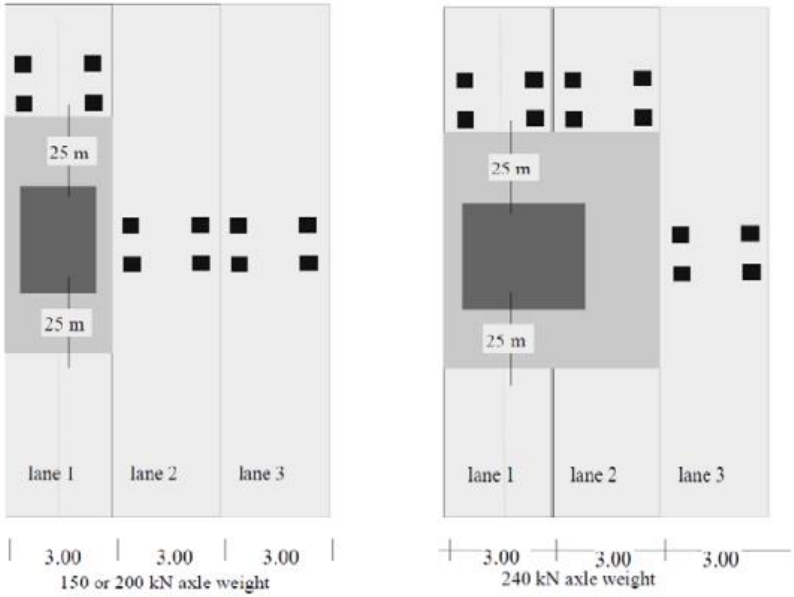


3. Load Model 3 - Special Vehicles

3. Load Model 3 - Special Vehicles



Arrangement of special vehicle on the carriageway



Simultaneity of special vehicles and load model n. 1

4. Load Model 4 - Crowd Loading

Distributed load of 5 kN/m2

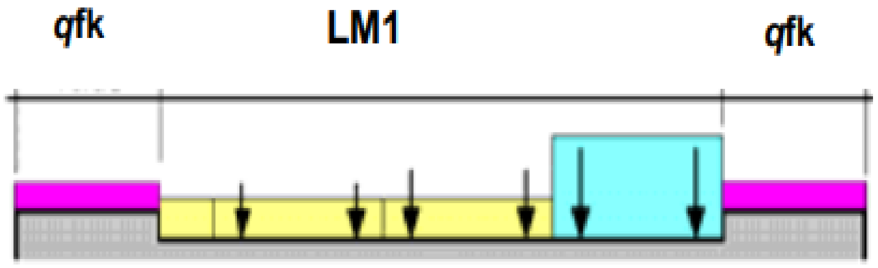
Moving Load Combinations

		CARRIAGEWAY						FOOTWAYS AND CYCLE TRACKS
Load type		Vertical forces				Horizontal forces		Vertical forces only
Reference		4.3.2	4.3.3	4.3.4	4.3.5	4.4.1	4.4.2	5.3.2-(1)
Load system		LM1 (TS and UDL systems)	LM2 (Single axle)	LM3 (Special vehicles)	LM4 (Crowd loading)	Braking and acceleration forces	Centrifugal and transverse forces	Uniformly Distributed load
Groups of Loads	gr1a	Characteristic values				a)	a)	Combination value b)
	gr1b		Characteristic value					
	gr2	Frequent values b)				Characteristic value	Characteristic value	
	gr3 d)							Characteristic value c)
	gr4				Characteristic value			Characteristic value b)
	gr5	See Annex A		Characteristic value				
	Dominant component action (designated as component associated with the group)							

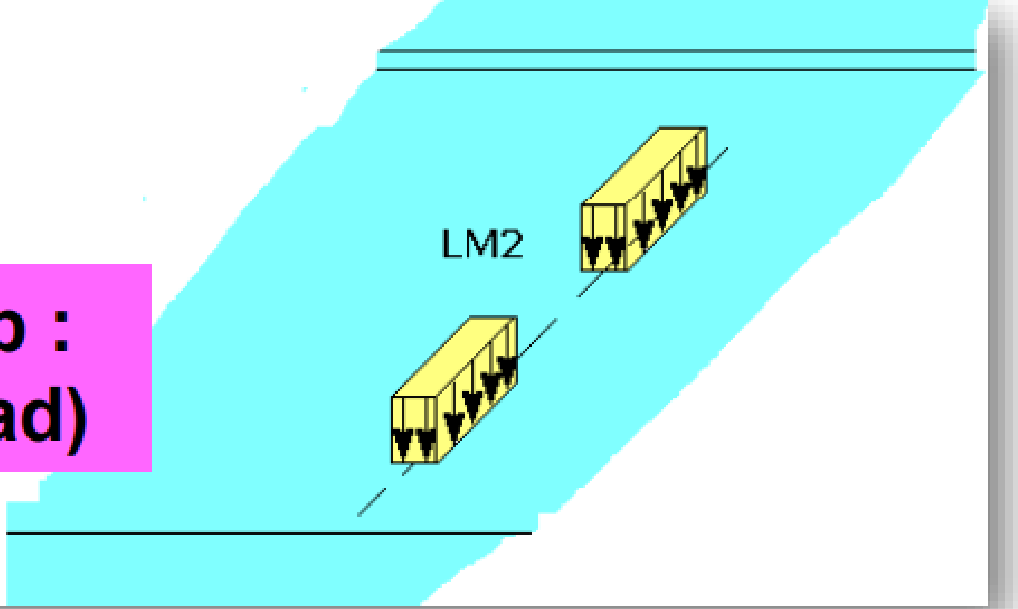
a) If specified, may be defined in the National Annex.
b) May be defined in the National Annex. Recommended value : 3 kN/m^2.
c) See 5.3.2.1-(3). One footway only should be considered to be loaded if the effect is more unfavourable than the effect of two loaded footways.
d) This group is irrelevant if gr4 is considered.

Group 1 a

Group of loads gr1a : LM1 + *combination value* of pedestrian load on footways or cycle tracks



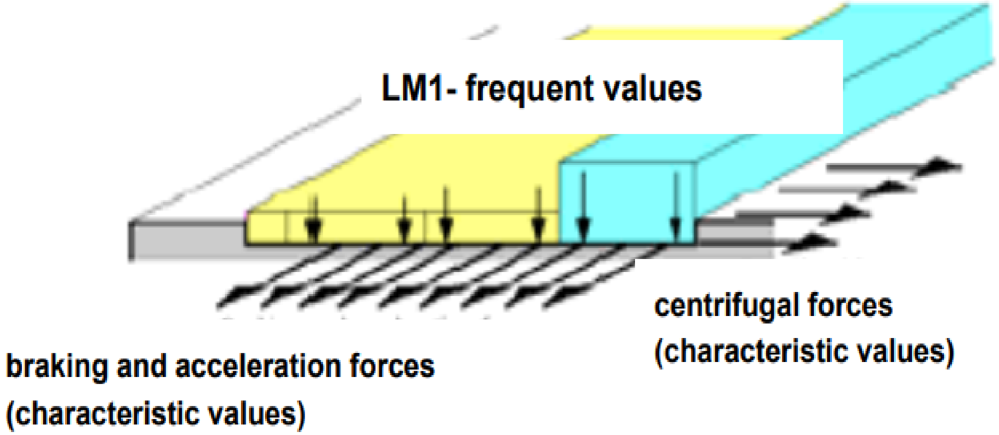
Group 1 b



Group of loads gr1b :
LM2 (single axle load)

Group 2

Group of loads gr2 :
***characteristic values* of horizontal forces,**
***frequent values* of LM1**



Group 3

Group of loads gr3 : loads on footways and cycle tracks

Group 4

Group of loads gr4 : crowd loading

Group 5

Group of loads gr5 : special vehicles (+ special conditions for normal traffic)

BRAKING LOADS

1. Braking Load for LM 1

i) for LM 1

Braking load is given by *Eq. 4.6, EN 1991-2:2003*

$$Q_{lk} = 0{,}6\alpha_{Q1}(2Q_{1k}) + 0{,}10\alpha_{q1}q_{1k}w_l L \qquad (4.6)$$
$$180\alpha_{Q1}\,(kN) \le Q_{lk} \le 900\,(kN)$$

α	=	Adjustment factor
Q	=	Axle Load
q	=	UDL
w	=	Total width of CW
L	=	Width of a Lane

BACKGROUND TO UK NATIONAL ANNEXES TO
EN 1990 AnnexA2 and EN1991-2

Table 1: Comparison of α-factors

EN1991-2		NAD		National Annex	
TS kN	UDL kN/m²	TS kN	UDL kN/m²	TS kN	UDL kN/m²
1.0	1.0	0.844	0.40	1.0	0.6111

BRAKING LOADS

2. Braking Load for SPV

ii) for LM 3 (SV196) — *Clause NA.2.18.1, NA to BS EN 1991-2:2003*

Braking Load, $Q_{lk,S}$ = δw (where δ = 0.25 for SV196)

Sl no	Axle load, w (kN)	δw (kN)
1	165	41.3
2	165	41.3
3	165	41.3
4	165	41.3
5	165	41.3
6	165	41.3
7	165	41.3
8	165	41.3
9	165	41.3
10	180	45.0
11	180	45.0
12	100	25.0
Total		486
UDL for total length of bridge		7.07 kN/m

Longitudinal braking and acceleration forces

The longitudinal force should be taken as the more severe of either the braking or the acceleration force, determined as below.

The characteristic value of longitudinal braking force on individual axles, $Q_{lk,S}$, expressed in kN, of special vehicles (both *SV* and *SOV*) should be calculated as follows:

$$Q_{lk,S} = \delta w$$

Where δ is the deceleration factor and w is the basic axle load of the relevant *SV* or *SOV* vehicle in *kN* shown in Figures NA.1, NA.2 and NA.3. The value of δ should be taken as 0,5 for *SV80*, 0,40 for *SV100*, 0,25 for the *SV196* and 0,20 for all of the *SOV* model vehicles.

Live Load Surcharge

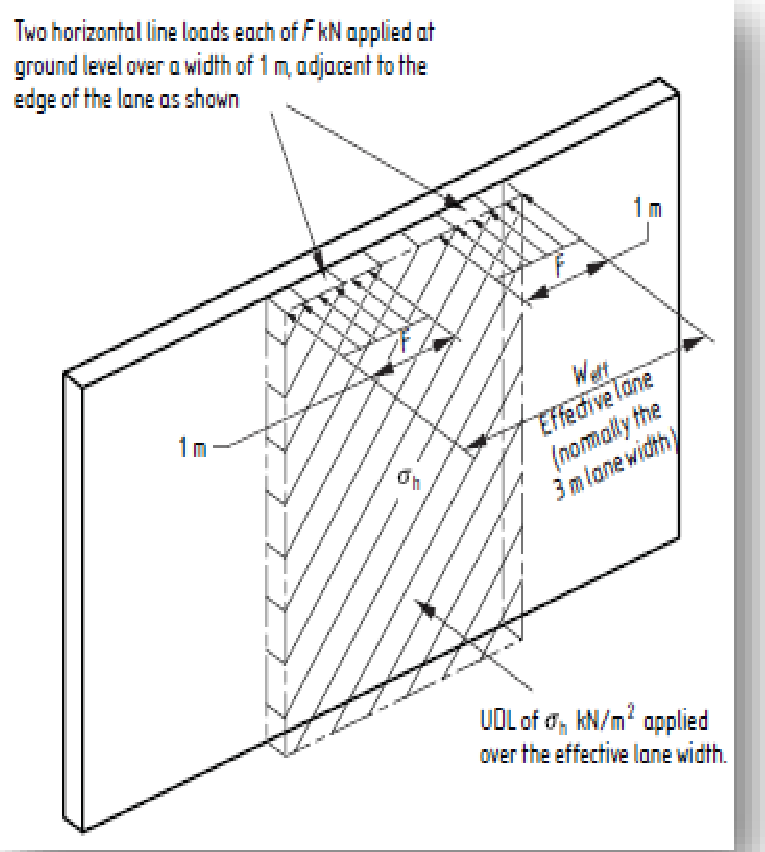


Table 7 Horizontal surcharge model for highway traffic loading

	Loading from normal traffic		SV/100 and SV/196 [A)]	
	Horizontal line load F at top of structure [B)]	Horizontal uniformly distributed load (UDL) σ_h	Horizontal line load F at top of structure [B)]	Horizontal UDL σ_h
Case A: Characteristic horizontal surcharge for each effective lane of traffic [C), D)]	Two 1 m wide line loads each $330K_dL_f$ (kN)	3 m wide UDL of $20K_dRL_f$ (kN/m^2)	Two 1 m wide line loads each $330K_d$ (kN)	3 m wide UDL of $30K_d$ (kN/m^2)
Case B: Characteristic horizontal surcharge for a metre width design [E)]	$330K_dD_f$ (kN/m) width	$20K_dR$ (kN/m^2)	$330K_d\,D_f$ (kN/m) width	$30K_d$ (kN/m^2)
Case C: Characteristic horizontal surcharge for segmental structures [F)]	$330K_d$ (kN/m) width	$20K_dR$ (kN/m^2)	$330K_d$ (kN/m) width	$30K_d$ (kN/m^2)

where:

K_d is the design value of K_a or K_o (as appropriate) based on Φ'_d

R is $3{,}0/W_{eff}$;

L_f is the lane factor specified in the UK National Annex to BS EN 1991-2:2003, NA.2.34.2; and

Uniform Temperature Load

BS EN 1991-1-5: 2003

EN 1991-1-5: 2003 (E)

Section 6 Temperature changes in bridges

6.1 Bridge decks

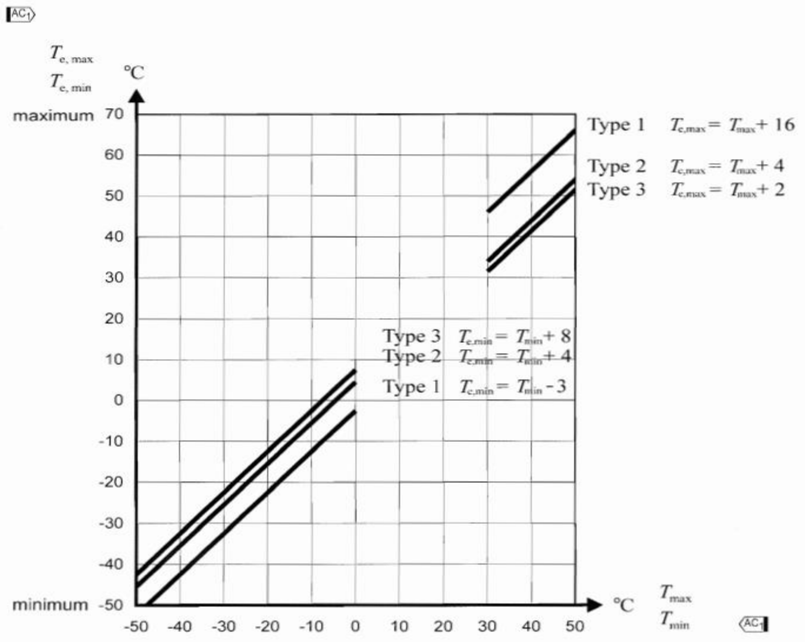
6.1.1 Bridge deck types

(1) For the purposes of this Part, bridge decks are grouped as follows:

Type 1	Steel deck:	- steel box girder - steel truss or plate girder
Type 2	Composite deck	
Type 3	Concrete deck:	- concrete slab - concrete beam - concrete box girder

BS EN 1991-1-5: 2003

EN 1991-1-5: 2003 (E)



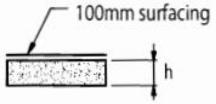
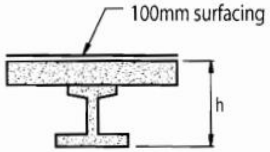
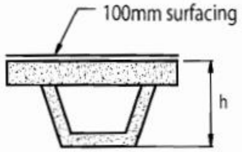
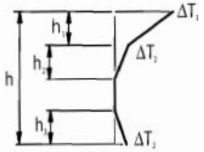
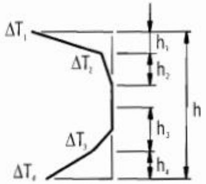
Uniform Temperature Load

Tmax=	40	(DBR)				
Tmin=	15					
For Type 3 Temax=	42		The minimum and maximum uniform			
Temin=	23		bridge temperature components			
T initial Temp for expansion T0e=	40	-	20	=	20	
T initial Temp for contraction T0c=	15	+	20	=	35	
Tn exp=	Temax	-	T0e			
=	22					
Tn cont=	Temax	-	T0c			
=	12					
considering long term effect, Tn exp=		11		(Midas input)		
Tn cont=		6				

AC1) NOTE 2: For bearings and expansion joints the National Annex may specify the maximum expansion range of the uniform bridge temperature component, and the maximum contraction range of the uniform bridge temperature component, if no other provisions are required. The recommended values are $(\Delta T_{N,exp} + 20)$ °C and $(\Delta T_{N,con.} + 20)$ °C, respectively. If the

Differential Temperature Load

BS EN 1991-1-5: 2003
EN 1991-1-5: 2003 (E)

Type of Construction	Temperature Difference (ΔT) (a) Heating	Temperature Difference (ΔT) (b) Cooling
 3a. Concrete slab  3b. Concrete beams  3c. Concrete box girder		

(a) Heating

$h_1 = 0.3h$ but ≤ 0.15m
$h_2 = 0.3h$ but ≥ 0.10m but ≤ 0.25m
$h_3 = 0.3h$ but $\le (0.10$m + surfacing depth in metres)
(for thin slabs, h_3 is limited by $h - h_1 - h_2$)

h	ΔT_1	ΔT_2	ΔT_3
		°C	
≤0.2	8.5	3.5	0.5
0.4	12.0	3.0	1.5
0.6	13.0	3.0	2.0
≥0.8	13.0	3.0	2.5

(b) Cooling

$h_1 = h_4 = 0.20h$ but ≤ 0.25m
$h_2 = h_3 = 0.25h$ but ≥ 0.20m

h	ΔT_1	ΔT_2	ΔT_3	ΔT_4
		°C		
≤0.2	-2.0	-0.5	-0.5	-1.5
0.4	-4.5	-1.4	-1.0	-3.5
0.6	-6.5	-1.8	-1.5	-5.0
0.8	-7.6	-1.7	-1.5	-6.0
1.0	-8.0	-1.5	-1.5	-6.3
≥1.5	-8.4	-0.5	-1.0	-6.5

Figure 6.2c: Temperature differences for bridge decks - Type 3 : Concrete Decks

*Note: The temperature difference ΔT incorporates ΔT_M and ΔT_E (see 4.3) together with a small part of component ΔT_N; this latter part has been included in the uniform bridge temperature component (see 6.1.3).

Seismic Load

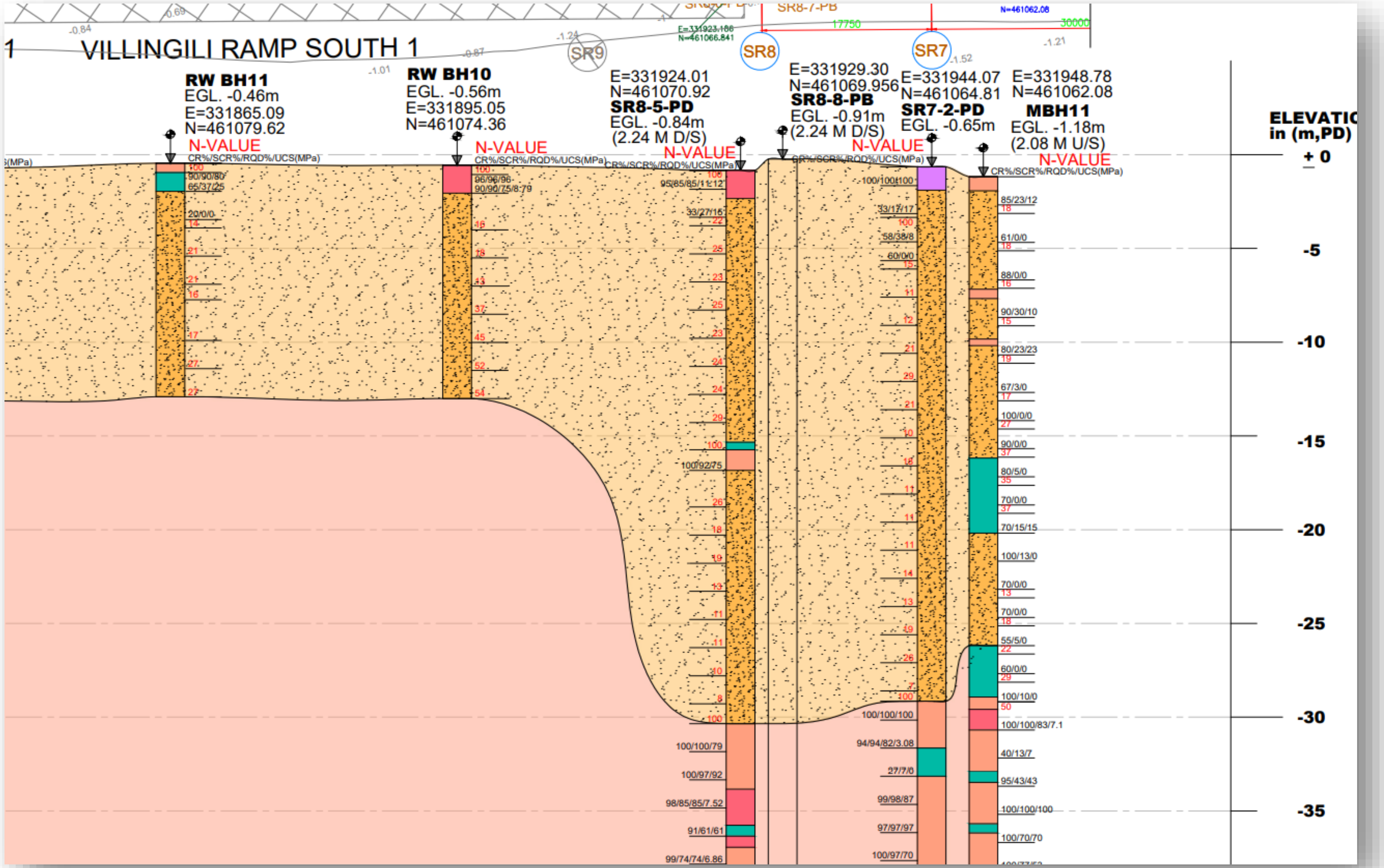
- Type of Ground – Type C (Depends on SPT values)
- Type of Response Spectra – Type 1 (Depends on Surface Wave Magnitude)

BS EN 1998-1:2004
EN 1998-1:2004 (E)

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70

Seismic Load



[Geotechnical Data](#)

Seismic Load

- Get the Values of S, Tb, Tc and Td

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

$S_e(T)$ is the elastic response spectrum;

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

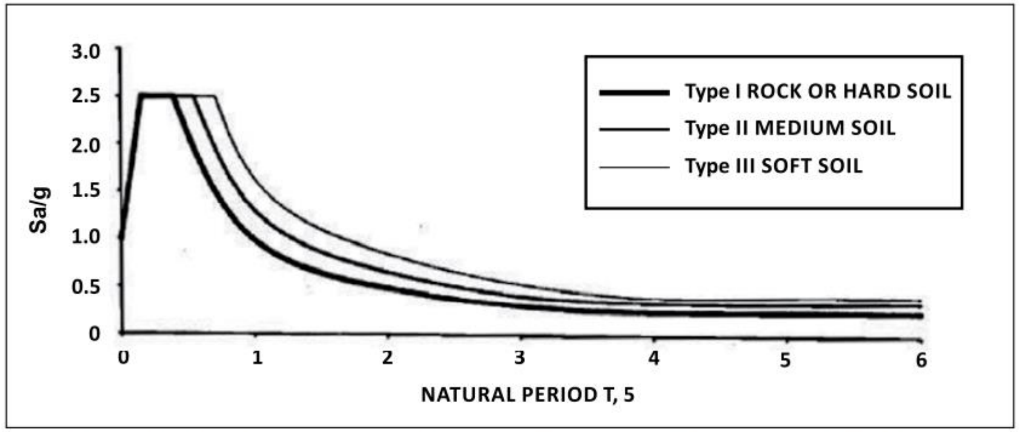


Fig. 5.1 (b) Spectra for Elastic Response Spectrum Method

Seismic Load

3.2.2.2 Horizontal elastic response spectrum

(1)P For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions (see Figure. 3.1):

$$0 \le T \le T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \quad (3.2)$$

$$T_B \le T \le T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \quad (3.3)$$

$$T_C \le T \le T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T}\right] \quad (3.4)$$

$$T_D \le T \le 4s : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C T_D}{T^2}\right] \quad (3.5)$$

where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_I . a_{gR}$);

T_B is the lower limit of the period of the constant spectral acceleration branch;

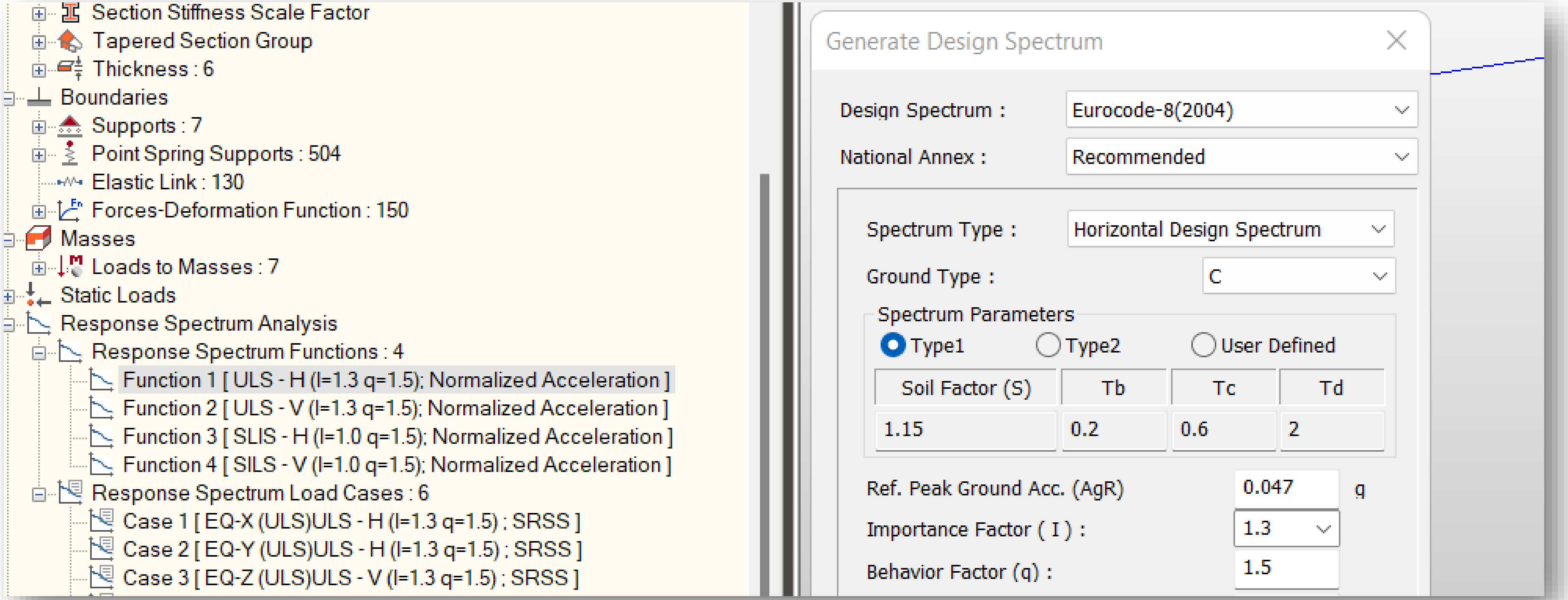
T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

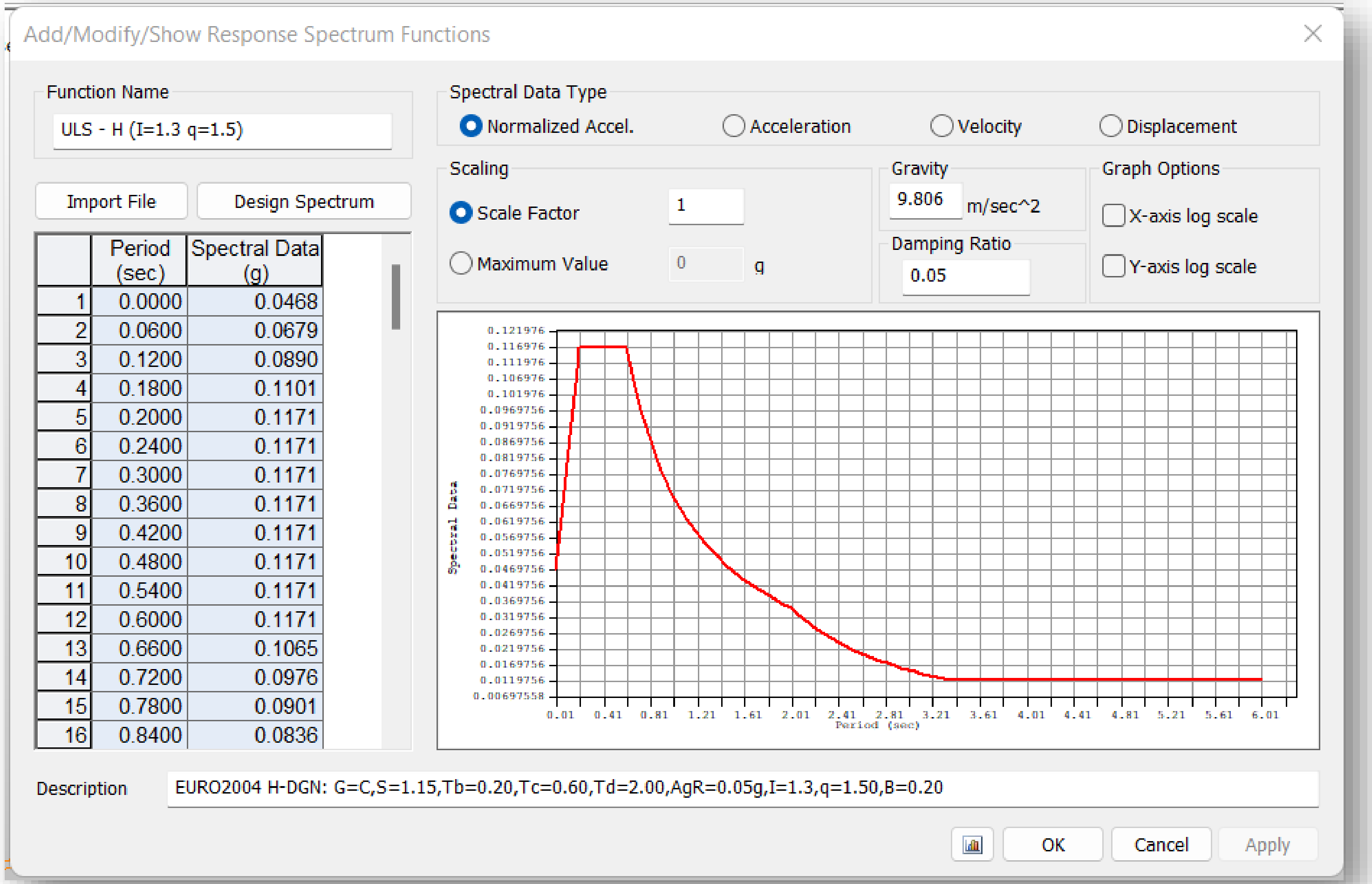
η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping, see **(3)** of this subclause.

Seismic Load



- **Fig** : Midas Window for Plotting Seismic Curve

Seismic Load



- **Fig** : Midas Window showing Seismic Curve

Wind Load

Basic Velocity Pressure $q_b = 1/2 . \rho . V_b^2$

Basic Wind Velocity, $V_b = C_{dir} . C_{season} . V_{b0}$

irrespective of wind direction and time of year (i.e. c_{dir} and c_{season} are both 1.0)

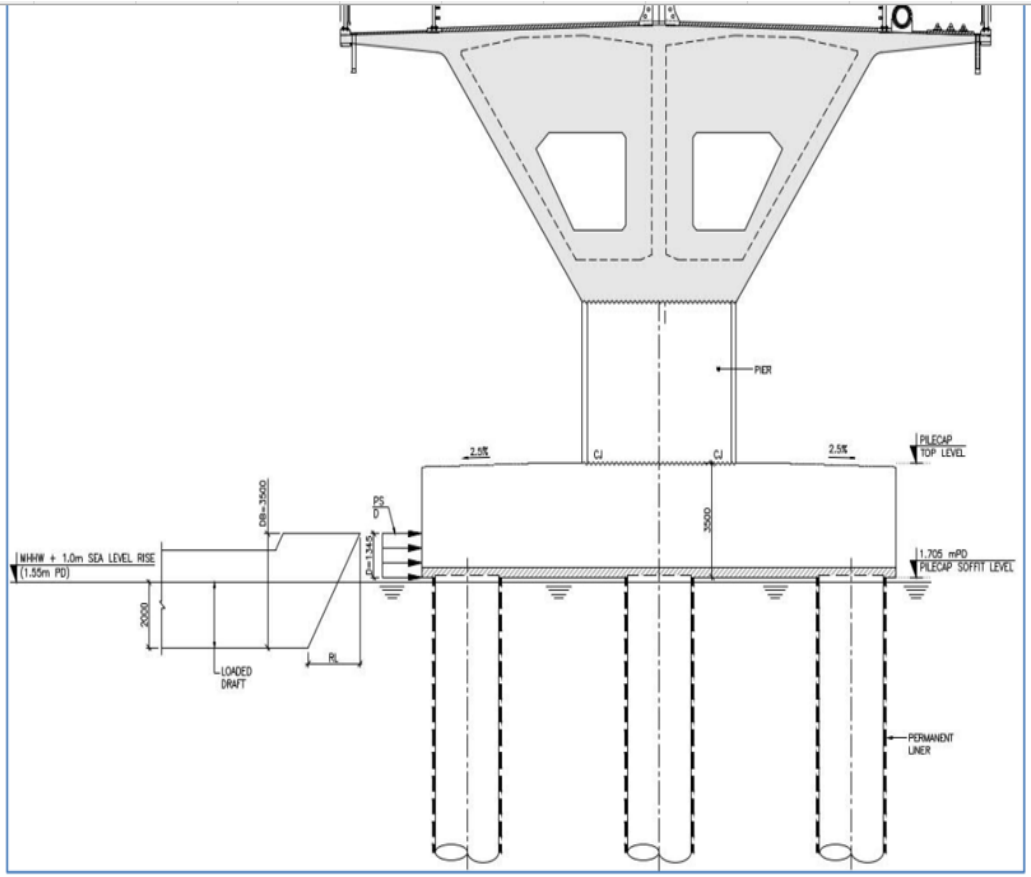
- Basic Wind Speed - **<u>27.7 m/s</u>**
- Height of the Deck from MSL - **<u>8m</u>**
- Drag coefficient – 1.3 (Fig. 8.3 EN 1991-1-4:2005)
- Exposure coefficient – 2.9 (Fig. NA.7, BS-EN 1991-1-4:2005)

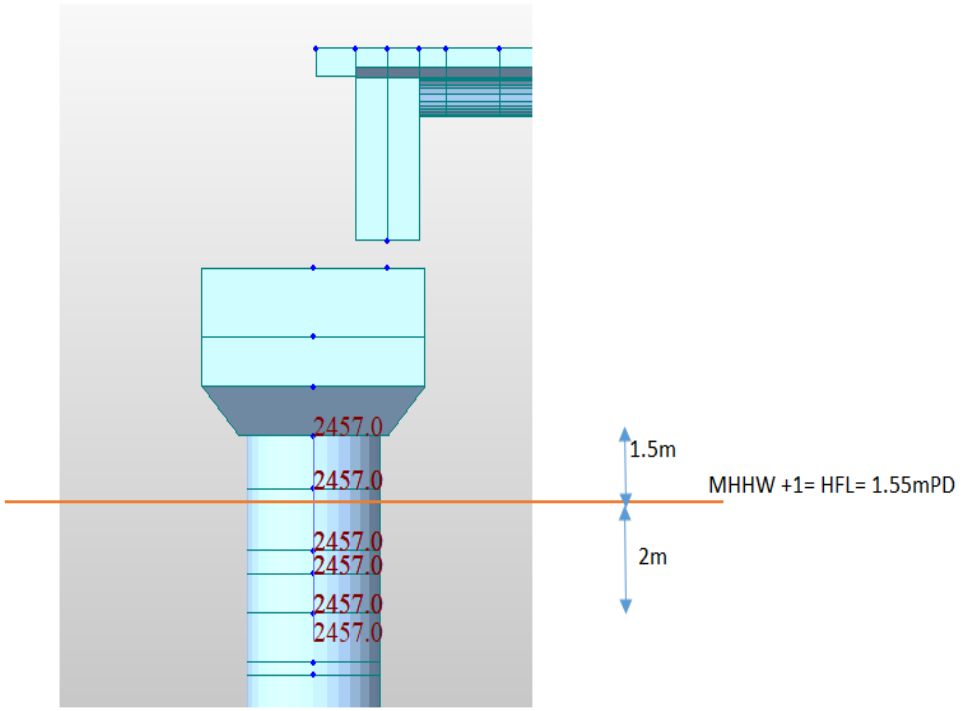
Ship Imapct

All the marine piers where water depth is more than 2m during MHHW, shall be designed for the ship impact design force shown in table below. The design vessel is based on the passenger ferries of 250 DWT, 35m LOA, 2m draft and 3.5m bow depth. The yearly mean current speed is assumed to be 3.5 m/s.

Distance of pier from centreline of each navigation channel [m]	Impact Force (including dynamic amplification factor of 1.3) [kN]
50	13800
60	12900
70	11900
80	11000
90	10000
100	9100
105	8600
>105	8600 (passenger ferries)

Ship Imapct





	=	2457 x	3.5
Ship Impact	=	8600 kN	

Figure 7: Case a) Loaded Barge Impact - Water at MHHW + 1m sea level rise

Tsunami Load

Tsunami force is calculated according to ASCE 7-16.

Tsunami Force = Fw = $0.5\ \rho_s\ I_{tsu}\ C_d\ b\ (h_e\ u^2)$

Tsunami pressure at certain depth = $0.5\ \rho_s\ I_{tsu}\ C_d\ b\ (u^2)$

ρ_s = $1.0\ \rho_{sw}$ = 1025 kg/m^3

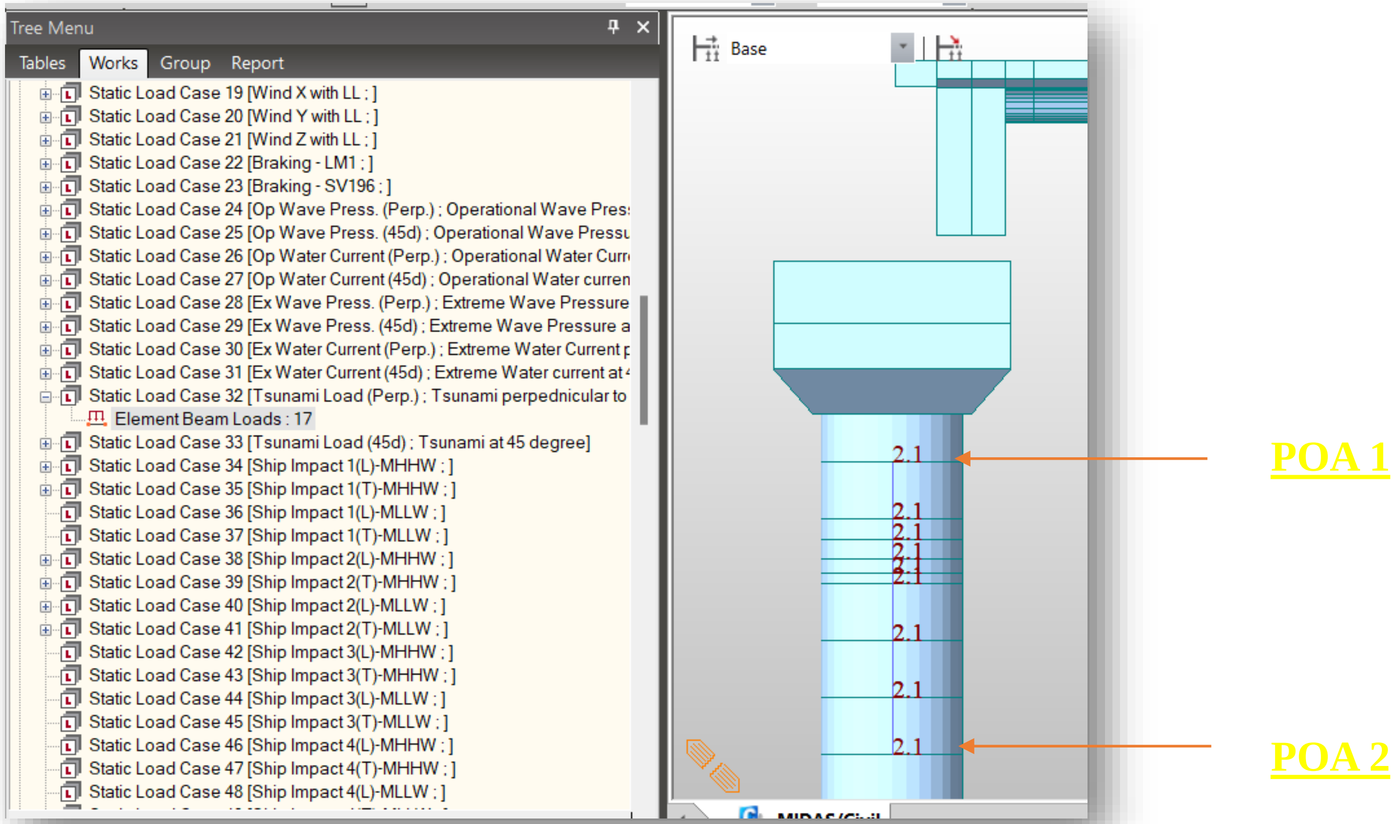
I_{tsu} = 1.0 Cl. 10.8.4 of Appendix B of Employers requirements

C_d = 1.2 Table 6.10-2-ASCE 7-16

u = Flow velocity

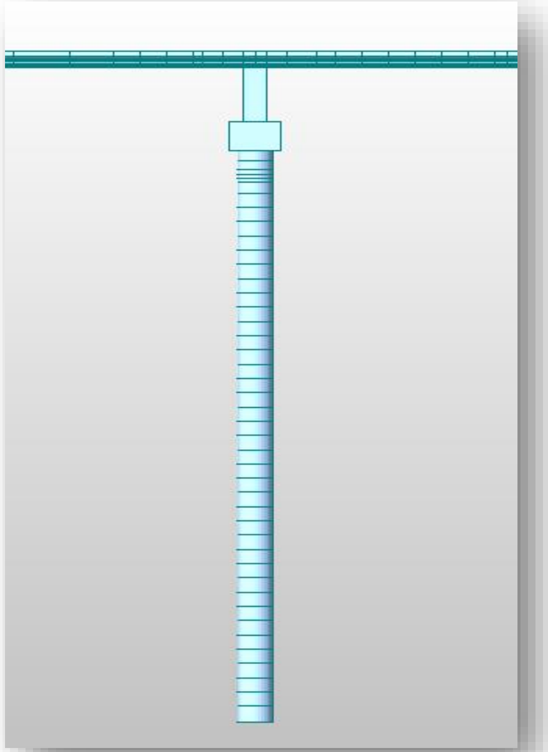
Tsunami Load- Point of Application

Point of application -	
POA 1 -	Tsunami water level= Tidal water level + Sea level rise + Tsunami amplitude
POA 2 -	Scour level

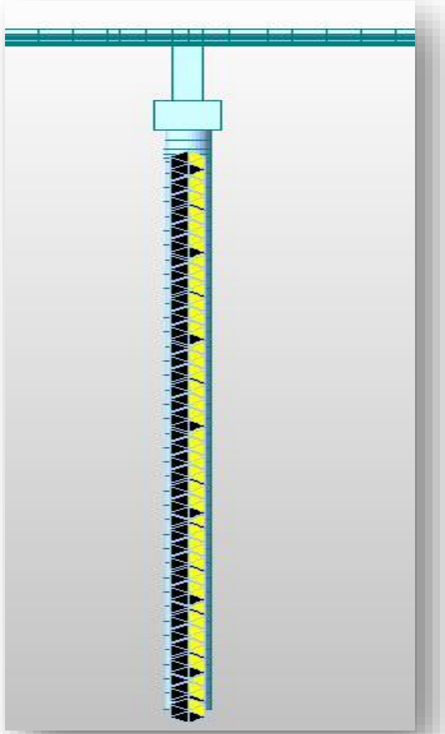


Soil Structure Interaction

Need of Soil Structure Interaction / P-Y curves

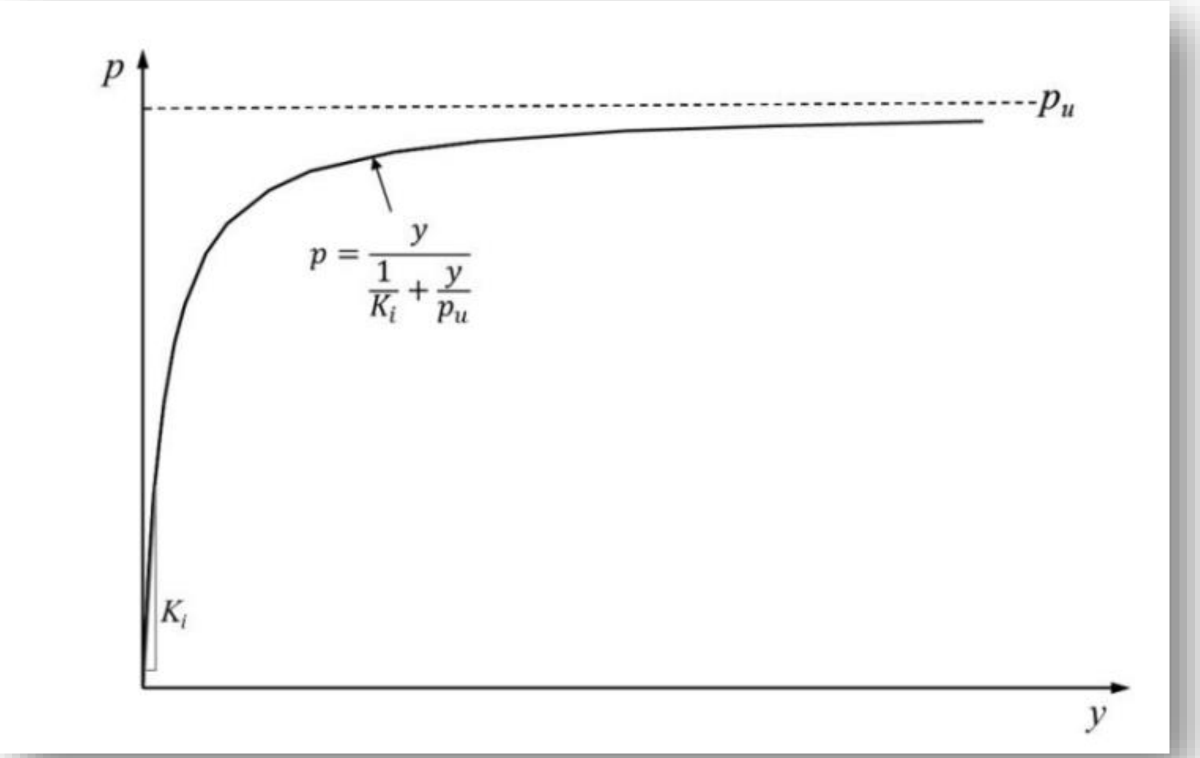


- Structure becomes more Rigid
- Time Period Decreases
- Seismic Forces Increases



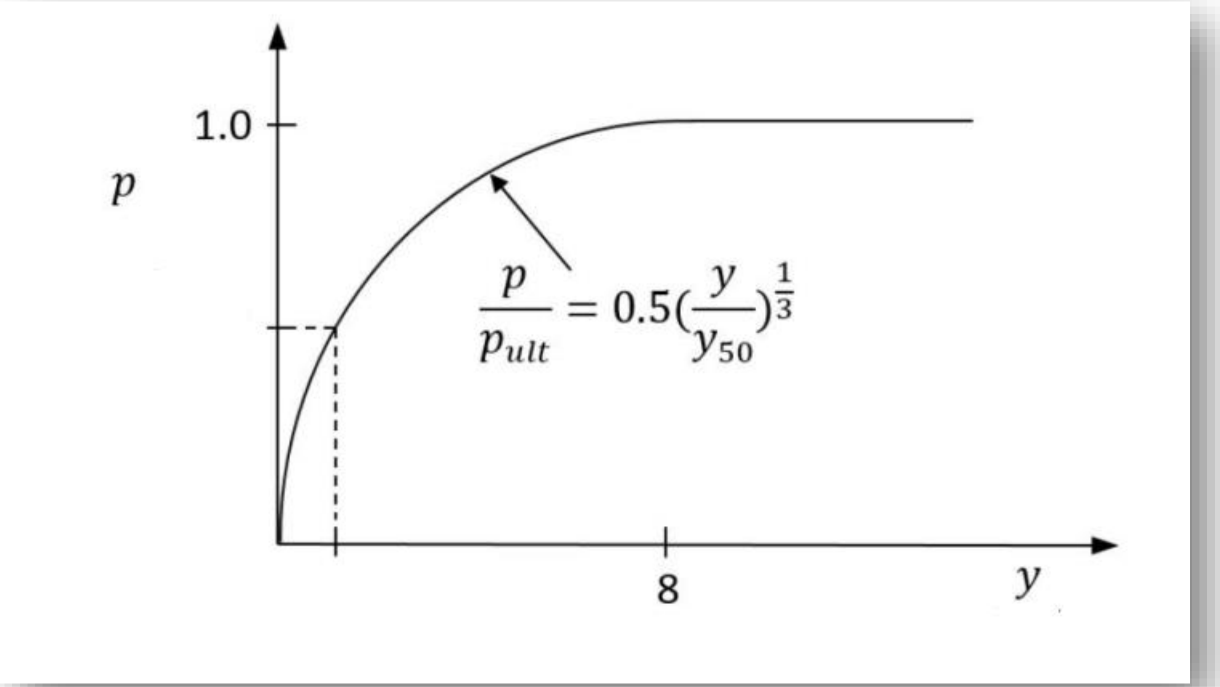
- Structure becomes more Flexible
- Time Period Increases
- Seismic Forces Decreases

P-Y Curves (Load – Deformation curve)



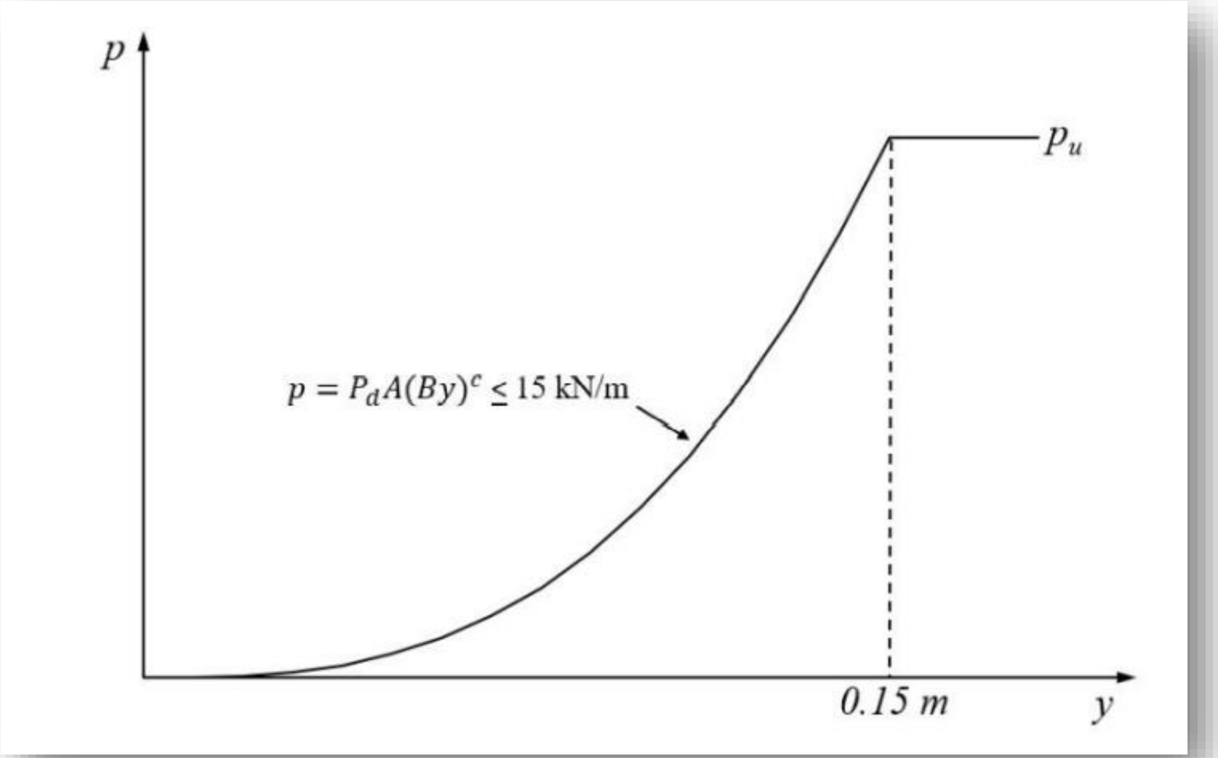
P-Y curve for Massive Rock

P-Y Curves



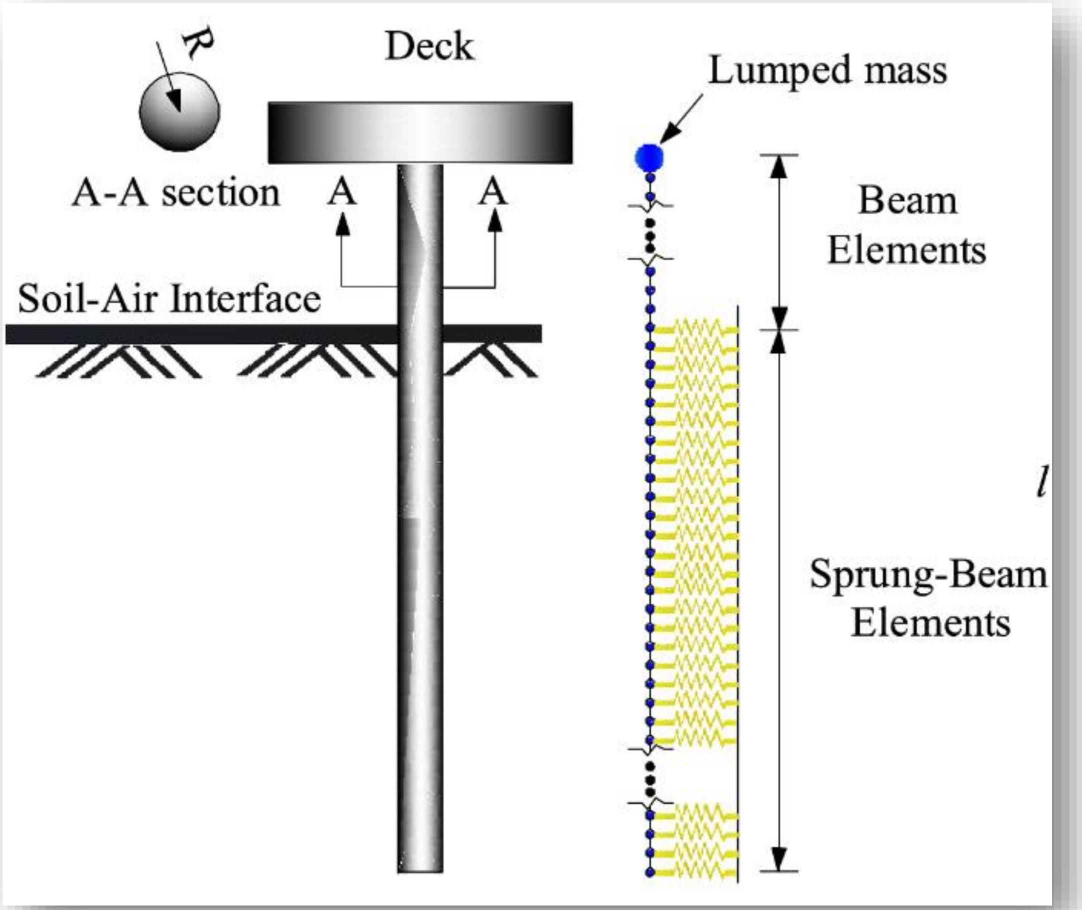
P-Y curve for Soft Clay

P-Y Curves



P-Y curve for Liquified Sands

Monopile Foundation



- A foundation consisting of a single, generally large-diameter, structural element that supports the entire load of a large above-surface structure.

Advantages - Monopile Foundation

Group Pile Option

Monopile Option

- Less space is required for construction
- Reduction in Number of Piles
- Less Construction Time
- No Differential settlement
- More Clearance for Navigational Spans
- In Mumbai Coastal road project number of piles has come down to 176 from 704 due to the introduction of Monopile system

Monopile Foundation

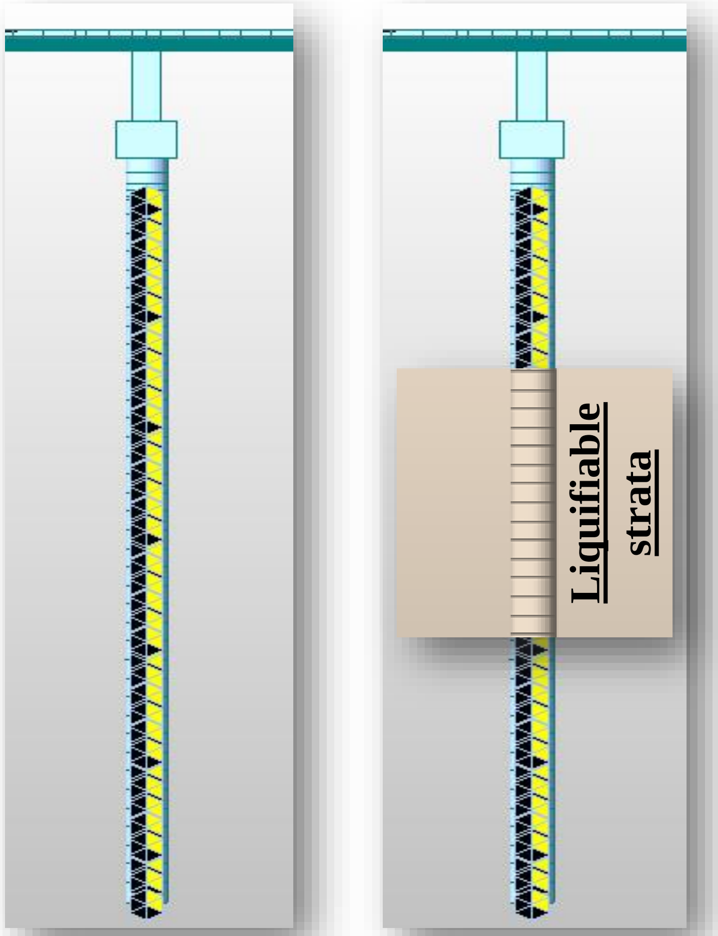


Fig : Liquifaction Effect

Design of Monopile Foundation

Design of Monopile

BS EN 1992-1-1:2004
EN 1992-1-1:2004 (E)

5.8.3 Simplified criteria for second order effects

5.8.3.1 Slenderness criterion for isolated members

(1) As an alternative to 5.8.2 (6), second order effects may be ignored if the slenderness λ (as defined in 5.8.3.2) is below a certain value λ_{lim}.

Note: The value of λ_{lim} for use in a Country may be found in its National Annex. The recommended value follows from:

$$\lambda_{lim} = 20 \cdot A \cdot B \cdot C / \sqrt{n} \quad (5.13N)$$

where:

A = $1 / (1+0,2\varphi_{ef})$ (if φ_{ef} is not known, A = 0,7 may be used)

B = $\sqrt{1+2\omega}$ (if ω is not known, B = 1,1 may be used)

C = $1,7 - r_m$ (if r_m is not known, C = 0,7 may be used)

φ_{ef} effective creep ratio; see 5.8.4;

ω = $A_s f_{yd} / (A_c f_{cd})$; mechanical reinforcement ratio;

A_s is the total area of longitudinal reinforcement

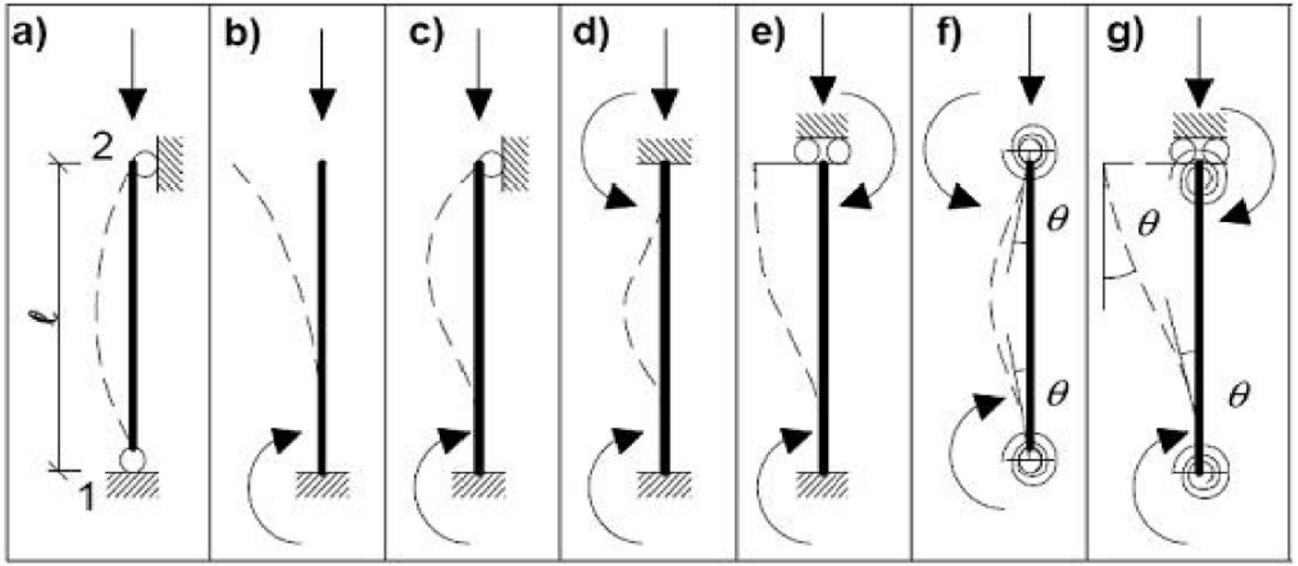
n = $N_{Ed} / (A_c f_{cd})$; relative normal force

r_m = M_{01}/M_{02}; moment ratio

M_{01}, M_{02} are the first order end moments, $|M_{02}| \geq |M_{01}|$

Design of Monopile

BS EN 1992-1-1:2004
EN 1992-1-1:2004 (E)



a) $l_0 = l$ b) $l_0 = 2l$ c) $l_0 = 0{,}7l$ d) $l_0 = l / 2$ e) $l_0 = l$ f) $l/2 < l_0 < l$ g) $l_0 > 2l$

Effective Length = Length of Pier from soffit level to pilehead top + pile head depth
+ Length of pile till depth of fixity

Design of Monopile

BS EN 1992-1-1:2004
EN 1992-1-1:2004 (E)

5.8.5 Methods of analysis

(1) The methods of analysis include a general method, based on non-linear second order analysis, see 5.8.6 and the following two simplified methods:

(a) Method based on nominal stiffness, see 5.8.7

(b) Method based on nominal curvature, see 5.8.8

Note 1: The selection of Simplified Method (a) and (b) to be used in a Country may be found in its National Annex.

Note 2: Nominal second order moments provided by the simplified methods (a) and (b) are sometimes greater than those corresponding to instability. This is to ensure that the total moment is compatible with the cross section resistance.

(2) Method (a) may be used for both isolated members and whole structures, if nominal stiffness values are estimated appropriately; see 5.8.7.

(3) Method (b) is mainly suitable for isolated members; see 5.8.8. However, with realistic assumptions concerning the distribution of curvature, the method in 5.8.8 can also be used for structures.

Design of Monopile

BS EN 1992-1-1:2004
EN 1992-1-1:2004 (E)

5.8.7.3 Moment magnification factor

(1) The total design moment, including second order moment, may be expressed as a AC1 magnification of the bending moments resulting from a first order analysis, namely: AC1

$$M_{Ed} = M_{0Ed}\left[1 + \frac{\beta}{(N_B / N_{Ed}) - 1}\right] \quad (5.28)$$

where:

M_{0Ed} is the first order moment; see also 5.8.8.2 (2)

β is a factor which depends on distribution of 1st and 2nd order moments, see 5.8.7.3 (2)-(3)

N_{Ed} is the design value of axial load

N_B is the buckling load based on nominal stiffness $= \frac{\pi^2 EI}{L^2}$

5.8.7.2 Nominal stiffness

(1) The following model may be used to estimate the nominal stiffness of slender compression members with arbitrary cross section:

$$EI = K_c E_{cd} I_c + K_s E_s I_s \quad (5.21)$$

where:

E_{cd} is the design value of the modulus of elasticity of concrete, see 5.8.6 (3)

I_c is the moment of inertia of concrete cross section

E_s is the design value of the modulus of elasticity of reinforcement, 5.8.6 (3)

I_s is the second moment of area of reinforcement, about the centre of area of the concrete

K_c is a factor for effects of cracking, creep etc, see 5.8.7.2 (2) or (3)

K_s is a factor for contribution of reinforcement, see 5.8.7.2 (2) or (3)

THANK YOU