STRUCTURES DESIGN MANUAL
FOR HIGHWAYS AND RAILWAYS

2013 Edition

AMENDMENT NO. 2/2023

December 2023

Highways Department

The Government of the Hong Kong Special Administrative Region

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INTRODUCTION

The “Structures Design Manual for Highways and Railways – 2013 Edition” (SDM) published by the Government of the Hong Kong Special Administrative Region sets out standards and provides guidance for the design of highway and railway structures in Hong Kong. In 2023, Highways Department conducted reviews on design requirements to facilitate routine inspection and maintenance of external prestressing tendons, climate change effects on highway structures, and design detail of lift shaft.

Following the reviews, amendments to Chapter 3, 5, 12 and 16 of the SDM are made.

AMENDMENT DETAILS

The following amendments are made :-

1. **CONTENTS**
   Pages 6, 13, 14, 15, 16 and 18 of the SDM are replaced by Replacement Sheets 1 to 6 respectively.

2. **CHAPTER 3**
   Pages 41, 42, 43, 43A, 44, 46 and 47 of the SDM are replaced by Replacement Sheets 7 to 13 respectively. Pages 54 to 56 of the SDM are replaced by Replacement Sheets 15 to 17 respectively. Page 47A is added by Replacement Sheet 14.

3. **CHAPTER 5**
   Pages 90, 90A, 90B and 90C of the SDM are replaced by Replacement Sheets 18 to 21 respectively.

5. **CHAPTER 12**
   Page 149 of the SDM is replaced by Replacement Sheet 22.

6 **CHAPTER 16**
   Pages 209 to 215 of the SDM are replaced by Replacement Sheets 23 to 29 respectively. Page 216 of the SDM is added by Replacement Sheet 30.

STRUCTURES DESIGN MANUAL

for Highways and Railways

2013 Edition

This electronic file is for reference only. When the content of this electronic file is inconsistent with the hard copy of the Structures Design Manual for Highways and Railways (SDM), 2013 Edition and the Amendments issued, the hard copy of the SDM, 2013 Edition and the Amendments shall prevail.

(This Version is Continuously Updated to include Amendments issued)

This Electronic File has incorporated the following Amendments:-

Rev	Issue Date	Amendment Incorporated
First Issue	May 2013	-
1	June 2018	Amendment No. 1/2018
2	April 2020	Amendment No. 1/2020
3	May 2021	Amendment No. 1/2021
4	January 2023	Amendment No. 1/2023
5	December 2023	Amendment No. 2/2023

Highways Department

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ρ = air density (taken as 1.226 kg/m^3).

By interpolating from the values for Waglan Island in Table 3.6, the above expression gives a value of 3.8 kN/m² for the peak velocity pressure corresponding to the maximum peak wind velocity of 79 m/s for a 120-year return period at an exposed location.

(4) All bridges should be assessed whether a "dynamic response procedure" as stated in BS EN 1991-1-4 Clause 8.2(1) is needed. The design procedure for such assessment should be in accordance with the UK NA to BS EN 1991-1-4 Clause NA.2.49 and PD 6688-1-4 Annex A, with the local parameters given in Clause 3.4.4 of this Manual.

(5) For bridges need not be designed with a "dynamic response procedure", two methods are provided for the calculation of wind actions on structures. The simpler requirements of Clause 3.4.2 may be applied for the majority of highway structures and railway bridges in Hong Kong. Clause 3.4.3 is to be used for structures where an enhanced level of overall structural reliability against failure from wind loading is desired and, shall be applied for all bridges meeting either of the following criteria :

(a) bridges with any span greater than 100m;

(b) bridges on Strategic Routes and as designated by the Chief Highway Engineer/Bridges and Structures; or AMD. 2/2023

(c) bridges with height above ground greater than 40m.

(6) A designer experiencing difficulty in deciding on the applicable clause for wind actions or an appropriate degree of exposure for a particular site shall consult the Chief Highway Engineer/Bridges and Structures for advice.

(7) Background information on the derivation of the wind actions given in this Manual is included in Appendix C for reference.

3.4.2 Simplified Procedure for Determining Peak Velocity Pressure

3.4.2.1 Peak Velocity Pressure for Wind Leading Combinations

(1) The peak velocity pressure q_p shall be obtained from Table 3.7 for bridges designed to the simplified procedure, regardless of the height of the structure above ground.

Table 3.7 – Peak Velocity Pressure q_p for Simplified Procedure

Sheltered Location	Exposed Location
q_p (kN/m^2)	**q_p (kN/m^2)**
2.5	3.8

(2) The values of peak velocity pressure to be used at locations of intermediate exposure are to be interpolated, by the use of engineering judgment, between the extremes given for sheltered and exposed locations in Table 3.7. To aid designers in choosing suitable values, descriptions and examples of typical locations are given in Table 3.8.

Table 3.8 – Exposure to Wind – Simplified Procedure

Degree of Exposure	Description	Peak Velocity Pressure q_p (kN/m^2)	Example
1	Sheltered by surrounding buildings and /or topography	2.5	Kowloon Park Drive Flyover
2	Normal exposure	2.8	Castle Road Flyover
3	Elevated situation; not sheltered by buildings or topography	3.3	Tai Po Road Interchange
4	Exposed to north-easterly or south easterly winds across open sea	3.8	Ap Lei Chau Bridge

(3) In general, for the design of walkway covers, sign gantries and noise barriers/enclosures, adoption of the simplified procedure to determine the peak velocity pressure q_p shall suffice. The peak velocity pressure q_p obtained from Table 3.7 or Table 3.8 shall be reduced by 20% (i.e. ranging from 2.0 kN/m^2 for sheltered location to 3.0 kN/m^2 for exposed location). If other methods are adopted, the designer shall consult the Chief Highway Engineer/Bridges and Structures for advice. AMD. 1/2023 AMD. 1/2020

(4) For the design of sign gantries/noise barriers/enclosures mounted on bridges, the peak velocity pressure can be reduced by 20% as mentioned in Clause 3.4.2.1 (3). However, for the design of bridges supporting the sign gantries/noise barriers/enclosures, the peak velocity pressure acting on sign gantries/noise barriers/enclosures and bridges shall be obtained from Table 3.7 or Table 3.8 without reduction. AMD. 1/2023

3.4.2.2 Peak Velocity Pressure for Traffic Leading Combinations

(1) For road bridges, the probability of much traffic being present on a bridge at peak wind velocity exceeding 44 m/s is low and the corresponding peak velocity pressure of 1.2 kN/m^2 may be used in traffic leading combinations. Therefore, as discussed in BS EN 1991-1-4 Clause 8.1(4), the combination value $\psi_0 F_{wk}$ of the wind action on the bridge and on the vehicles travelling on the bridge, should be limited to a value F_W^* determined by taking q_p as 1.2 kN/m^2. Provision given in the UK NA to BS EN 1991-1-4 Clause NA.2.47 shall not be followed.

(2) For railway underbridges, the value of q_p for determining F_W^{**}, which is discussed in BS EN 1991-1-4 Clause 8.1(5), shall be agreed with the appropriate railway authority taking into account the possibility of the presence of railway traffic on the bridge at high wind velocity.

3.4.2.3 Climate Change Effects on Simplified Procedure

Under a research for wind effects on bridges due to climate change completed in 2023, it is noted that the wind speed is expected to increase in future due to climate change. It is also noted that the peak velocity pressure for simplified procedure is derived from the wind data collected from the wind station of Waglan Island, which has a degree of conservatism for the design of conventional bridge structures. It is concluded that the peak velocity pressure specified in Tables 3.7 and 3.8 for simplified procedure can provide an adequate margin for covering the climate change effects for ordinary highway structures, and should remain unchanged. **AMD. 2/2023**

3.4.3 Full Procedure for Determining Peak Velocity Pressure

For bridges to be designed under the full procedure, due account shall be taken of the loaded length under consideration and the height of the structure above ground. Provisions given in this Section shall replace Clause NA.2.56 of the UK NA to BS EN 1991-1-4.

3.4.3.1 Peak Velocity Pressure for Wind Leading Combinations

The peak velocity pressure $q_p(z)$ shall be taken as follows :

$q_p(z) = q_{pb}(z)\ K_{pc}$

The hourly mean velocity pressure $q'(z)$ shall be taken as follows :

$q'(z) = q_b'(z)\ K_{pc}$

where $q_{pb}(z)$ is the basic peak velocity pressure obtained from Table 3.9 appropriate to the height of the bridge and loaded length under consideration; **AMD. 2/2023**

$q_b'(z)$ is the basic hourly mean velocity pressure obtained from Table 3.9 appropriate to the height of the bridge and loaded length under consideration; and

K_{pc} is the climate change velocity pressure multiplying factor defined in Clause 3.4.3.6.

3.4.3.2 Velocity Pressure on Relieving Areas for Wind Leading Combinations

Where wind on any part of a bridge or element gives relief to the member under consideration, the effective coexistent value of velocity pressure on the parts affording relief shall be determined in accordance with Clause 3.4.3.1 as the appropriate hourly mean velocity pressure $q'(z)$. **AMD. 2/2023**

3.4.3.3 Peak Velocity Pressure for Traffic Leading Combinations

The peak velocity pressure $q_p(z)$ on those parts of the bridge or its elements on which the application of wind actions increases the effect being considered shall be taken as :

(1) For road bridges, $q_p(z)$ determined in accordance with Clause 3.4.3.1 shall be adopted, but the combination value $\psi_0 F_{wk}$ of the wind action on the bridge and on the vehicles travelling on the bridge, which is discussed in BS EN 1991-1-4 Clause 8.1(4), should be limited to a value $F_W{}^*$ determined by taking $q_p(z)$ as $q'(z)$ determined in accordance with Clause 3.4.3.1. Provision given in the UK NA to BS EN 1991-1-4 Clause NA.2.47 shall not be followed. **AMD. 2/2023**

(2) For railway underbridges, the value of $q_p(z)$ for determining $F_W{}^{**}$, which is discussed in BS EN 1991-1-4 Clause 8.1(5), shall be agreed with the appropriate railway authority taking into account the possibility of the presence of railway traffic on the bridge at high wind velocity.

3.4.3.4 Velocity Pressure on Relieving Area for Traffic Leading Combinations

Where wind on any part of a bridge or element gives relief to the member under consideration, the effective coexistent value of velocity pressure $q_L'(z)$ on the parts affording relief shall be taken as:

$q_L'(z) = 1.2\, q'(z) / q_p(z)$

where $q'(z)$ and $q_p(z)$ are determined in accordance with Clause 3.4.3.1. **AMD. 2/2023**

Table 3.9 – Basic Peak Velocity Pressure $q_{pb}(z)$ and Basic Hourly Mean Velocity Pressure $q_b'(z)$ for Full Procedure

Height z above ground level (m)	Basic Peak Velocity Pressure $q_{pb}(z)$ (kN/m^2) appropriate to horizontal wind loaded lengths (m)							Basic Hourly Mean Velocity Pressure $q_b'(z)$ (kN/m^2)
	20	100	200	400	600	1000	2000	
10	4.2	2.8	2.4	2.1	2.0	1.8	1.6	0.8
15	4.2	2.8	2.5	2.2	2.0	1.9	1.7	0.9
20	4.2	2.8	2.5	2.2	2.1	2.0	1.8	1.0
30	4.2	2.9	2.6	2.3	2.3	2.1	1.9	1.1
40	4.2	3.0	2.7	2.5	2.3	2.2	2.0	1.3
50	4.2	3.1	2.8	2.5	2.4	2.3	2.1	1.4
60	4.3	3.1	2.9	2.6	2.5	2.4	2.2	1.5
80	4.3	3.3	3.0	2.8	2.7	2.5	2.4	1.7
100	4.4	3.4	3.1	2.9	2.8	2.7	2.5	1.8
150	4.6	3.6	3.4	3.2	3.1	3.0	2.8	2.1
200	4.8	3.8	3.6	3.4	3.3	3.2	3.0	2.3

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Notes: (1) For locations which are less exposed, as described in Table 3.8 according to the degree of exposure, the values $q_{pb}(z)$ and $q_b'(z)$ given above may be factored according to the degree of exposure as follows :

Degree of exposure	Factor of $q_{pb}(z)$ and $q_b'(z)$
1	0.7
2	0.8
3	0.9
4	1.0

(2) The horizontal wind loaded length shall be that giving the most severe effect. Where there is only one adverse area (see BS EN 1991-2) for the element or structure under consideration, the wind loaded length is the base length of the adverse area. Where there are more than one adverse area, as for continuous construction, the maximum effect shall be determined by consideration of any one adverse area or a combination of adverse areas, using the $q_{pb}(z)$ appropriate to the base length of the total combined base lengths. The remaining adverse areas, if any, and the relieving areas, are subjected to wind having a velocity pressure as given in Clause 3.4.3.2 for wind leading combinations and in Clause 3.4.3.4 for traffic leading combinations.

(3) Where the bridge is located at an orography significant site, the $q_{pb}(z)$ and $q_b'(z)$ shall be factored by $(c_o(z))^2$ where $c_o(z)$ is a topographical factor. For the definition of significant orography, Clause 4.3.3 of BS EN 1991-1-4 and Clause NA.2.13 of the UK NA to BS EN 1991-1-4 shall be referred. $c_o(z)$ for $q_b'(z)$ shall be determined based on Annex A.3 of BS EN 1991-1-4, and $c_o(z)$ for $q_{pb}(z)$ shall be taken as follows:

$c_o(z) = 1$ for $\Phi < 0.05$

$$= 1 + 2s\phi \frac{s_c(z)}{s_b(z)} \quad \text{for } 0.05 < \Phi < 0.3$$

$$= 1 + 0.6s \frac{s_c(z)}{s_b(z)} \quad \text{for } \Phi > 0.3$$

where

s = orographic location factor as given in Annex A.3 of BS EN 1991-1-4
Φ = upwind slope as defined in Annex A.3 of BS EN 1991-1-4
$s_b(z)$ = terrain and bridge factor as given in Table 3.10
$s_c(z)$ = hourly velocity factor as given in Table 3.10

(4) Vertical elements such as piers and towers shall be divided into strips in accordance with the heights given in column 1 of this table and the $q_{pb}(z)$ shall be derived from the centroid of each unit.

(2) For live load producing a relieving effect, the ULS partial factor γ_Q shall be taken as 1.0.

3.4.3.6 Climate Change Velocity Pressure Multiplying Factor for Full Procedure

As stated in Clause 3.4.1(5), the full procedure is applied to specific highway structures which require an enhanced level of overall structural reliability against failure from wind loading. As the wind speed is expected to increase in future due to climate change, the basic peak velocity pressure $q_{pb}(z)$ and basic hourly mean velocity pressure $q_b'(z)$ specified in Table 3.9 shall be multiplied by a climate change velocity pressure multiplying factor K_{pc} in order to determine the peak velocity pressure $q_p(z)$ and hourly mean velocity pressure $q'(z)$. The climate change velocity pressure multiplying factor K_{pc} shall be taken as follows : **AMD. 2/2023**

$K_{pc} = 1.22$

3.4.4 Dynamic Response Procedure

(1) Criteria given in Clause NA.2.49 of the UK NA to BS EN 1991-1-4 and PD 6688-1-4 Annex A for verifying the need of "dynamic response procedure" shall be followed. To account for the different wind environment in Hong Kong, the modifications given in Clauses 3.4.4.1 and 3.4.4.2 shall be made.

(2) Some background information on the aerodynamic effects on bridges is provided in Appendix D of this Manual for the designer's reference.

(3) When wind tunnel tests are required, the provisions given in Clause A.5 of PD 6688-1-4 and the further guidance on wind tunnel tests given in Appendix E of this Manual shall be followed.

3.4.4.1 General Substitutions

The site mean wind velocity $v_m(z)$ adopted in the UK NA Clause NA.2.49 and PD 6688-1-4 Annex A shall be replaced by the hourly mean wind velocity for relieving area $v_r(z)$, which shall be derived at the appropriate height of the bridge above ground level, z, in metres. The values for use in Hong Kong shall be taken as :

$v_r(z) = v_s\, s_c(z)\, c_o(z)$ **AMD. 2/2023**

where $v_s = v_{sb}\, K_{vc}$

v_{sb} is the basic hourly mean wind velocity at 10 m height and shall be taken as 35 m/s for all sites in Hong Kong. This is appropriate to a 120-year return period.

K_{vc} is the climate change velocity multiplying factor, which shall be taken as 1.106.

$s_c(z)$ is the hourly velocity factor given in Table 3.10.

$c_o(z)$ is the topographical factor given in Table 3.9.

3.4.4.2 Specific Substitutions

(1) The partial factors given in PD 6688-1-4 Clause A1.5.4.5 shall not be used, and shall be replaced by the partial factors γ_Q for wind actions given in Clause 3.2.1 of this Manual.

(2) The wind speed v_{WO} for verifying the divergent amplitude response required in PD 6688-1-4 Clauses A.2.4.2 and A.4.4.2, shall be taken as :

$$v_{WO} = K_{1U} K_{1A} \frac{[v_r(z) + 2v_d(z)]}{3}$$

where $v_d(z)$ is the peak wind velocity and shall be taken as :

$v_d(z) = v_s \, s_b(z) \, c_o(z)$

$v_r(z)$, v_s and $c_o(z)$ are defined in Clause 3.4.4.1

$s_b(z)$ is the terrain and bridge factor given in Table 3.10

K_{1U} is the factor to cover the uncertainty of prediction of wind velocity and shall be taken as 1.1

K_{1A} is the coefficient selected to give an appropriate low probability of occurrence of the severe forms of oscillation and shall be taken as 1.4.

(3) The wind speed criteria for wind tunnel testing required in PD 6688-1-4 Clause A.5, shall be taken as :

$$v_{WO} = K_{1U} K_{1A} \frac{[v_r(z) + 2v_d(z)]}{3}$$

$$v_{w\alpha} = K_{1U} K_{1A} v_r(z)$$

$$v_{WE} = K_{1U} K_{1A} \frac{[v_r(z) + v_d(z)]}{2}$$

where $v_r(z)$ is defined in Clause 3.4.4.1

$v_d(z)$, K_{1U} and K_{1A} are defined in Clause 3.4.4.2(2)

Angle of wind inclination $\alpha = \overline{\alpha} \pm 7\left[\dfrac{s_b(z)}{s_c(z)} - 1\right]$.

3.4.5 Wind Forces

(1) The simplified method given in Clause 8.3.2(1) of BS EN 1991-1-4 shall not be followed.

(2) The size factor c_s and the dynamic factor c_d, shall both be taken as 1.0 for bridges that a "dynamic response procedure" is not needed.

3.4.6 Force Coefficients and Pressure Coefficients

AMD. 1/2018

(1) In addition to the force coefficients given in Clause 8.3.1 of BS EN 1991-1-4 and Table NA.8 of the UK NA to BS EN 1991-1-4, the force coefficients given in Clauses 3.4.6.1 to 3.4.6.5 below shall be followed.

(2) The pressure coefficients for noise barriers and noise enclosures given in Clauses 3.4.6.6 and 3.4.6.7 below shall be followed. AMD. 1/2018

3.5 TEMPERATURE EFFECTS

3.5.1 General

(1) The provisions given in BS EN 1991-1-5 and the UK NA to BS EN 1991-1-5 for thermal actions on highway and railway structures shall be followed unless otherwise specified in this Manual.

(2) Daily and seasonal fluctuations in shade air temperature, solar radiation, re-radiation, etc. cause the followings :

(a) Changes in the uniform temperature of a bridge superstructure which, in turn, govern its movement. The uniform temperature is a theoretical temperature calculated by weighting and adding temperatures measured at various levels within the superstructure. The weighting is in the ratio of the area of cross-section at the various levels to the total area of cross-section of the superstructure. Over a period of time, there will be a minimum, a maximum, and a range of uniform bridge temperature, resulting in loads and/or load effects within the superstructure due to :

(i) restraint of associated expansion or contraction by the form of construction (e.g. portal frame, arch, flexible pier, elastomeric bearings) referred to as temperature restraint; and

(ii) friction at roller or sliding bearings where the form of the structure permits associated expansion and contraction, referred to as frictional bearing restraint.

(b) Differences in temperature between the top surface and other levels in the superstructure. These are referred to as temperature differences and they result in loads and/or load effects within the superstructure.

3.5.2 Uniform Temperature Components

(1) The minimum uniform bridge temperature $T_{e,min}$ and maximum uniform bridge temperature $T_{e,max}$ given in Figure 6.1 of BS EN 1991-1-5 and Clause NA.2.4 of the UK NA to BS EN 1991-1-5, and the minimum shade air temperature T_{min} and maximum shade air temperature T_{max} given in Clause NA.2.5 of the UK NA to BS EN 1991-1-5 shall not be used.

(2) Based on a research project completed in 2023 on the climate change effects on bridge temperatures, the maximum bridge temperature will increase due to climate change. The minimum and maximum uniform bridge temperatures ($T_{e,min}$ and $T_{e,max}$) for Hong Kong shall be obtained directly from Table 3.17 for superstructure Types 1 to 3 which have incorporated climate change effects. Basic uniform temperatures appropriate to normal structures shall be used except for the cases given in Clause 3.5.2(3). AMD. 2/2023

Table 3.17 – Uniform Bridge Temperature (with Climate Change Effects)

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Superstructure Type (see Figure 3.2 for classification)	Normal Structures		Minor Structures	
	$T_{e,min}$ °C	$T_{e,max}$ °C	$T_{e,min}$ °C	$T_{e,max}$ °C
1	0	55	0	53
2	0	48	0	46
3	0	45	0	43

(3) Basic uniform temperatures appropriate to minor structures may be used for :

(a) foot/cycle track bridges,

(b) carriageway joints and similar equipment likely to be replaced during the life of the structure, and

(c) erection loading.

(4) The uniform bridge temperatures are dependent on the depth of surfacing on the bridge deck, and the values given in Table 3.17 assume surfacing depths of 40 mm for Type 1 and 100 mm for Types 2 and 3. Where the depth of surfacing differs from these values, the minimum and maximum uniform bridge temperatures shall be adjusted by the amounts given in Table 3.18. Adjustments given in Table NA.1 of the UK NA to BS EN 1991-1-5 shall not be used.

Table 3.18 – Adjustment to Uniform Bridge Temperature for Deck Surfacing

	Additional To Minimum Uniform Bridge Temperature °C			Additional To Maximum Uniform Bridge Temperature °C		
Deck Surface	**Type 1**	**Type 2**	**Type 3**	**Type 1**	**Type 2**	**Type 3**
Unsurfaced Plain	0	-3	-1	+4	0	0
Unsurfaced Trafficked or Waterproofed	0	-3	-1	+2	+4	+2
40 mm Surfacing	0	-2	-1	0	+2	+1
100 mm Surfacing(1)	N/A	0	0	N/A	0	0
200 mm Surfacing(1)	N/A	+3	+1	N/A	-4	-2
Notes: (1) Surfacing depths include waterproofing (2) N/A = not applicable						

(5) The values of uniform temperature given in Table 3.17 shall be adjusted for height above mean sea level by subtracting 0.5ºC per 100 m height for minimum uniform temperatures and 1.0ºC per 100 m height for maximum uniform temperatures.

(6) The initial bridge temperature T_0 at the time the structure is effectively restrained upon completion of construction shall be taken as 30ºC for calculating contraction down to the minimum uniform bridge temperature component and 10ºC for calculating expansion up to the maximum uniform bridge temperature component. Values given in Clause A.1(3) NOTE of BS EN 1991-1-5 and Clause NA.2.21 of the UK NA to BS EN 1991-1-5 shall not be used.

3.5.3 Temperature Difference Components

(1) The effects of the vertical temperature differences within the superstructure shall be considered by including a non-linear temperature difference component, i.e. Approach 2 defined in Clause 6.1.4.2 of BS EN 1991-1-5. The value of the sum of the linear and non-linear part of the temperature difference, ΔT, shall be determined from the data given in Figure 3.2. Figures 6.2a to 6.2c of BS EN 1991-1-5 and Clause NA.2.9 of the UK NA to BS EN 1991-1-5 shall not be used.

(2) Positive temperature differences occur when conditions are such that solar radiation and other effects cause a gain in heat through the top surface of the superstructure. Conversely, reverse temperature differences occur when conditions are such that heat is lost from the top surface of the bridge deck as a result of re-radiation and other effects.

(3) Temperature differences are sensitive to the thickness of surfacing, and the data given in Figure 3.2 assume depths of 40 mm for Type 1 and 100 mm for Type 2 and 3. For other depths of surfacing, the values given in Table 3.19, Table 3.20 and Table 3.21 may be used as appropriate. Temperature differences given in Annex B of BS EN 1991-1-5 and Clause NA.2.23 of the UK NA to BS EN 1991-1-5 shall not be used.

(c) shrinkage and creep of concrete;

(d) friction and wobble;

(e) draw-in,

where appropriate giving details of any assumption made, and also making clear whether allowance shall be made for anchorage and jack losses.

(4) Consideration must be given at the design stage to the practicability of fitting one or other of the acceptable proprietary post-tensioning systems into the work being designed, so that the post-tensioning specialists are not set an impossible task. End-block reinforcement depends on the type of anchorage used, and so shall not be detailed, but, again, consideration shall be given at the design stage to likely requirements. The proposals submitted by the main contractor must accordingly include end-block reinforcement details.

5.6.3 External Prestressing

(1) All highway structures and railway bridges adopting external prestressing shall be checked to ensure that the removal or failure either of any two external tendons or 25% of those at any one section, whichever has the more onerous effect, will not lead to collapse at the ultimate limit state under the design ultimate permanent actions.

(2) External tendons that are not located inside the closed cells of box-girder bridge deck can be susceptible to fire and mechanical damages. Project-specific requirements and provisions for the protection of the tendons shall be proposed for agreement by the Chief Highway Engineer/Bridges and Structures and the respective maintenance authorities. AMD. 1/2020

(3) All external tendons shall be replaceable and provisions shall be made in the design for the de-tensioning, removal and replacement of any external tendon. The use of prestressing components of the types that would facilitate the de-tensioning, removal and replacement of the tendons, such as sheaths/ducts of double casing type at deviators/anchor diaphragms and provision of spare opening for tendon run-through at diaphragms and deviators, should be considered. Where appropriate, tendons with independent strands with individual HDPE sheaths etc. should be considered. AMD. 2/2023

(4) Where the detailing does not enable tendons to be removed and replaced without damage to either the tendons or the structure, a method statement defining in details how the tendons can be removed and replaced shall be provided. A method statement defining in details how the structure can be demolished shall also be provided. AMD. 1/2020

(5) Where it is necessary to restrict traffic on the highway structure to replace the tendons, the extent of this restriction shall be agreed with the relevant authorities and defined in a method statement. It should be noted that traffic restrictions may not be appropriate for highly utilized structures with high delay costs.

(6) Provisions shall also be made in the design to facilitate routine inspection, including close visual inspection (i.e. visual inspection which is carried out at touching distance), and maintenance of the tendons, particularly at the locations of deviators and anchor diaphragms where the tendons/anchors are encased in concrete and at the locations where tendons are located at or above the height of 2m. In particular, each tendon shall be provided with an identification tag at every span showing the tendon reference number and anchorages shall be fabricated with inspection holes located to permit a probe or inspection by borescope of the upper part of the duct behind the anchor heads. The holes shall also facilitate the post-grouting inspection. Alternatively, the anchorages shall be equipped with a device which permits the inspection personnel to monitor and verify the complete filling of grout in the anchorage. Anchorage caps covering the inspection holes shall be designed to be removable as necessary for access to the inspection holes. **AMD. 2/2023**

(7) A robust multiple barrier protection system shall be used to protect the external tendons from weathering and corrosion.

(8) For grouted tendons, consideration should be given to the use of vacuum-assisted grouting for improved quality of grouting especially for long horizontal tendons and for tendons without access/vents at the high points of the tendon profiles.

(9) Length of tendons shall not exceed 200m, and length between grout injection point and the most distant grout vent/anchor head shall not exceed 100m, unless it can be demonstrated with grouting trials that complete filling of the tendon ducts, with the tendons completely surrounded with grout, can be satisfactorily achieved. Similarly for tendons to be injected with other flexible corrosion-inhibiting products. **AMD. 1/2020**

(10) It is preferable to provide remote monitoring and warning system for the detection of tendon/strand/wire breakage. At locations where inspection of tendons is difficult, remote monitoring and warning system shall be provided. The need and details of the provision shall be agreed with the respective maintenance offices during the design stage.

(11) Detailed method statements describing the procedures and their purposes as well as the quality checking arrangement shall be approved by the Engineer before commencement of the prestressing works.

(12) To inspect the quality of grouting, hammer sounding or other equivalent inspection methods shall be conducted at all grouted tendons before handing over of the completed structures to the maintenance authority. If suspected voids are detected, further verification by local tendon duct sectioning or borescope inspection through drilled holes shall be carried out. **AMD. 1/2021**

(13) The quality of grouting at anchorages and deviators shall also be inspected. Such inspection can be conducted in form of visual inspections of the grouting condition at grout vents. If suspected abnormalities are found, further verification by borescope inspection through grout vents or pre-installed inspection holes shall be carried out

(14) The inspection of quality of grouting shall not be conducted by the prestressing works contractor or his agents, and shall be conducted by the site supervisory staff or other independent parties.

(15) Adequate training shall be provided to all supervisory personnel and workers to ensure their awareness of the purposes of every step and detail of the prestressing works.

(16) To facilitate future maintenance, the following records shall also be passed to the respective maintenance authorities upon handing over of completed structures:

(a) Information of prestressing system and components, such as product names, serial numbers, catalogues, materials, details of corrosion protection system, testing records, as-built dimensions and profiles.

(b) Records of grouting operations, including location, date and time, weather conditions, technical personnel supervising or carrying out the grouting operations, prestressing tendon reference numbers, grout mix, admixtures used, grouting equipment, grouting methods and procedures, actual locations of grout vents and taps, grout material test reports, grouting trial reports, air test of grout vents and detailed records of the grouting operation (such as injection pressures, volume of grout used, time and duration of grouting, and details of any interruptions and topping up).

(c) Records of tensioning operations, including location of the operations, coil, heat and bundle numbers of strand used, date and time, weather conditions, technical personnel supervising or carrying out tensioning operations, prestressing tendon reference numbers, tensioning apparatus identification, prestressing sequence, measured extensions, amount of draw-in and pressure gauge or load cell reading. **AMD. 1/2021**

(d) Records of duct friction tests.

(e) Other particulars, records and reports in relation to prestressing works which are required to be submitted by the contractors under the contract specifications.

(f) Records of hammer sounding inspection at tendons, the inspection of grouting condition at anchorages and deviators, and the subsequent remedial and reinstatements works. The inspection records shall be checked and signed by the project office or the resident site staff.

(g) Any abnormality observed during prestessing works.

(h) Method statements on tendon replacement/removal as mentioned in Clauses 5.6.3(4) and 5.6.3(5).

(i) Information on provisions made in the design to facilitate routine inspection and maintenance of tendons as mentioned in Clause 5.6.3(6).

(j) Design calculation and computer model.

(17) Where circumstances justify it, other external prestressing tendon systems comprising of individual strands, each with permanent protective materials and sheating, such as a monostrand system, may be considered in the design as alternatives to grouted tendons. If an alternative system is considered feasible, project-specific design and maintenance

requirements shall be proposed for the agreement by Chief Highway Engineer/Bridges and Structures and the respective maintenance authorities. **AMD. 1/2021**

(18) Bridge structures with external prestressing system shall also be designed for maintainability in accordance with Highways Department Guidance Notes No. NT/GN/050 – "Guidance Notes on Maintainability Requirements for Bridge Structures with External Prestressing Systems". In particular, a report on "Assessment of maintainability for future inspection, operation and maintenance of bridge structures with external prestressing system" shall be submitted to the maintenance authority for agreement during the preliminary and detailed design stage. **AMD. 2/2023**

5.6.4 Specialist Prestressing Contractors

(1) All prestressed concrete works for highway structures shall be carried out by specialist contractors in the Prestressed Concrete Works for Highway Structures Category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works.

(2) The Prestressed Concrete Works for Highway Structures Category consists of two classes:

Class I - Supply and Installation of Prestressing Systems; and

Class II - Supply of Prestressed Concrete Units.

(3) The supply and installation of on-site prestressing work shall be carried out by a contractor in Class I. Precast prestressed units manufactured off-site shall be supplied by a contractor in Class II.

(3) Floor finishes shall be non-slip complying with Clause 12.12. A tubular stainless steel handrail shall be provided on all sides (except sides with access door) of the lift car, extending to within 150 mm of all corners. The handrail shall be 950 mm above finished floor level.

(4) People waiting at lift entrance should not cause obstruction to the normal pedestrian flow. A lift waiting area with cover shall be provided at lift entrance.

(5) The horizontal clearance between a lift shaft enclosure and a road carriageway shall be in accordance with the TPDM. Barrier fences shall be provided and the lift structure shall be designed to resist collision loads.

(6) The lift shaft and lift car walls shall be glazed as appropriate to minimise visual impact and to facilitate security monitoring of the lift car interior. Glazing shall be clear toughened glass to provide resistance to damage by vandals and accidental breakage. Notwithstanding, lift glazing creates greenhouse effect and generates substantial energy demand for air conditioning and ventilation. To reduce the greenhouse effect, lift glazing shall be "low-emissivity" glass to lower the rate of solar heat transmission into the lift shaft and lift car. Due consideration should also be given to the following energy saving measures :

  (a) reduction of transparent areas subject to agreement by Highways Department, Transport Department, the police, etc but without jeopardizing the security monitoring of the lift car interior;

  (b) use of opaque walls on appropriate sides and orientating the remaining glazed sides to the south or east, where sunshine is less, if possible;

  (c) use of opaque roof top;

  (d) measures described in Clause 12.15.2;

  (e) other measures such as sun screen provision, as appropriate.

(7) The lower part of full-height glass panel in the lift car shall be translucent and provided with skirting.

(8) Access to clean interior surfaces of the glass panels of the lift shaft and lift car shall be provided.

(9) Provision shall be made for hoisting device including lifting beams or eyes within the lift shaft and the machine rooms. Lift machine rooms shall be above ground with good access for vehicles nearby and be provided with adequate artificial lighting and ventilation.

(10) In order to improve the cleanliness and hygiene conditions, the top surfaces of stem walls inside the lift shaft shall be inclined to avoid accumulation of dirt. **AMD. 2/2023**

16.1.3 Spare Parts

(1) Certain components are susceptible to damage or wear during the service life of a highway structure or railway bridge. A range of spare parts shall be provided for such components upon their handing over to the maintenance authority. Certain components such as tiles, cladding panels and special parapet railings are susceptible to damage or wear during the service life of a highway structure or railway bridge. A range of spare parts shall be provided for such components upon their handing over to the maintenance authority. The aim of this provision is to achieve economy in overall cost and to ensure proper maintenance. In general, for those components which (or equivalent) are readily available in local market, it is not cost effective to keep any more stocks. The maintenance authority shall be consulted at an early stage in the design of the structures for advice on the requirements for such.

(2) The project proponents, when requested by the maintenance authority, shall provide accommodation within the new structures to store the spare parts.

16.1.4 Specific Considerations for Bridge Structures with External Prestressing Tendons

(1) Access openings of not less than 800 mm wide by 1000 mm high shall be provided through all internal diaphragms and at end diaphragms for entry to deck void via bridge abutments. The access openings shall be clear of any obstruction or installation. Lockable doors shall be installed at the access openings at end diaphragms to restrict access by unauthorized personnel. The access openings shall be easily accessible and do not require temporary traffic arrangement. If such requirement is considered impractical under exceptional circumstance, the project proponent shall seek explicit written agreement from the maintenance authority.

(2) For structures with substantial lengths, such as sea viaducts, access openings for entry to deck void shall be provided at an interval of not more than 1 km along the bridge structures. Permanent access platforms extended from the roadside to the access openings shall be provided. **AMD. 2/2023**

(3) A maintenance corridor shall be provided for the passage of personnel and equipment within the deck cell. The maintenance corridor shall be an even and a continuous path with a minimum headroom of not less than 2 m and a clear width of not less than 800 mm. All temporary installations, such as temporary blisters to facilitate the erection of bridge structure, shall be removed after construction. For permanent installations, they shall be positioned properly to provide a clear maintenance corridor. Utilities and drainage installations shall be properly installed in the deck cell to avoid encroachment on the maintenance corridor. If such requirement(s) is/are considered impractical under exceptional circumstances, the project proponent shall seek explicit written agreement from maintenance authority.

16.2 SAFETY CIRCUITS FOR BRIDGES OVER NAVIGABLE CHANNELS

(1) Ships often collide with bridges over navigable channels. The severity of damage caused by such collisions depends on the design of the bridges, but fatalities can result from vehicles falling into waterways because their drivers are unaware of collision damage.

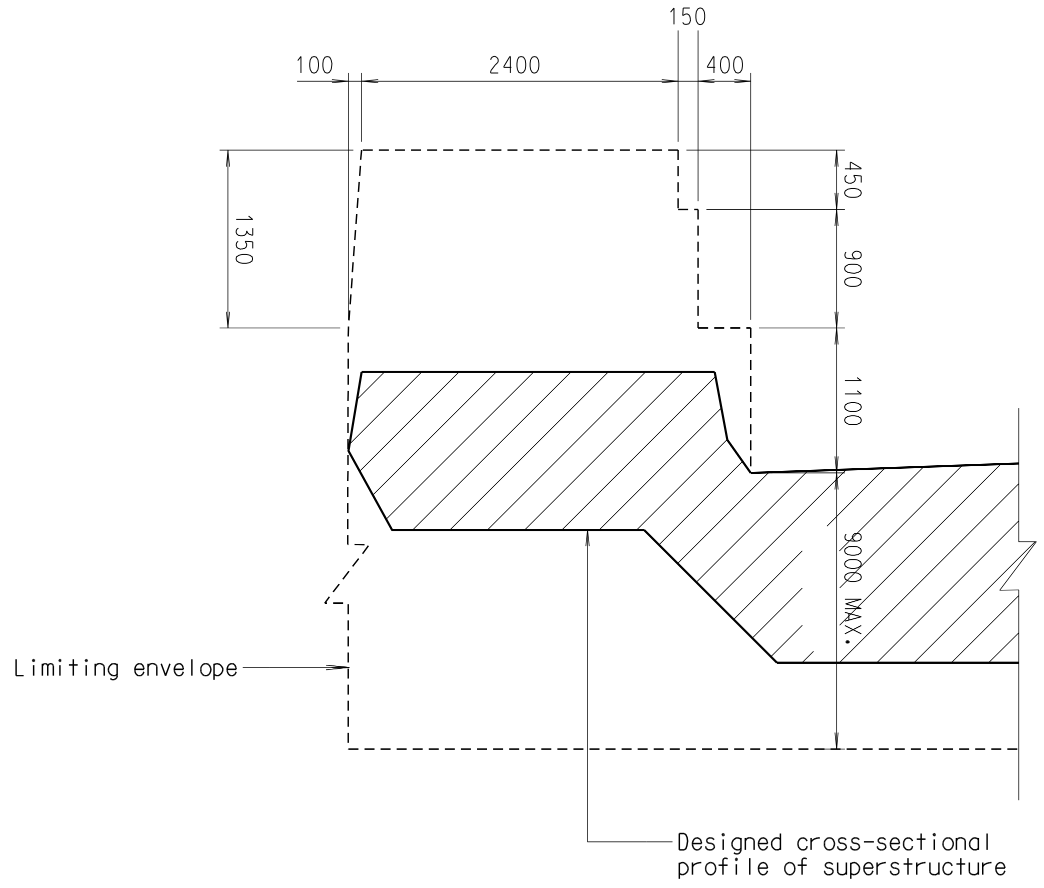
(2) Bridges exposed to ship collisions shall be provided with safety circuits which activate stop lights to halt passing traffic if the bridge is seriously damaged.

(3) Such bridges are also subjected to less severe blows from passing vessels which may not be severe enough to cause collapse but can result in a damage that may not be readily apparent and needs repairs. Secondary safety circuits capable of detecting and recording such blows shall be fitted to draw attention to the need for inspection and maintenance.

16.3 LIMITING ENVELOPE FOR STRUCTURAL ELEMENTS AND INSTALLATIONS

(1) To facilitate the use of the underbridge inspection vehicle for the inspection and maintenance of the underside of the structure, no part of the structure or any projection therefrom, except road lighting, fire hydrants, emergency telephones and noise barrier, shall protrude beyond the limiting envelope as defined by the shaded area in Figure 16.1.

Figure 16.1 – Limiting Envelop for Highway Structure



(2) The prior agreement of the maintenance authority shall be obtained at an early stage in the design of the structure for incorporating any continuous or isolated installations which protrude beyond this limiting envelope. Nevertheless any such continuous installations shall be easily removable and the clear distance between the isolated protruding elements which are not easily removable shall not be less than 2000 mm.

16.4 PAINTING OF STEELWORK

(1) All steelwork, including sign gantries and vehicle and pedestrian parapets, shall be either hot dip galvanized or metal sprayed and shall in addition be painted. As in all painting work, the surface to be painted shall be thoroughly cleaned and prepared, free from grease, dirt, scale and rust in accordance with the requirements of BS EN ISO 12944 Part 4.

(2) The volatile organic compounds (VOC) content for the paints shall be in compliance with the Air Pollution Control (Volatile Organic Compounds) Regulation (the Regulation), and shall not exceed the maximum limits of VOC content for the Regulated Architectural Paints as listed in the Regulation. As a general reference for highway works, paints classified as "Industrial Maintenance Coatings" with a VOC content limit of 250g/L under the Regulation are appropriate paint materials to be used for compliance.

(3) Direct application of paint to newly galvanized steelwork will result in premature failure of the paint system. Such failures are usually due to the formation of brittle zinc soaps at the paint/zinc interface with the resultant loss of adhesion and deterioration in the properties of the paint film. The pretreatment of the surface with a proprietary two pack etch primer prior to painting would prevent the failure of the paint system. Primers shall be applied in thin coats by continuous spraying and strictly in accordance with the manufacturer's instructions. Suitable one pack primers are also available, but care must be taken to ensure that they are formulated for use on galvanized steel.

(4) Weathering of galvanized surfaces until all bright zinc has changed to a dull surface by oxidation may aid adhesion of the paint, provided any loose particles have been removed from the surface. The deliberate use of weathering as a pretreatment for painting is not recommended as the minimum time needed for full weathering cannot easily be assessed. It may also be difficult to completely clean a weathered surface in preparation for painting.

(5) After galvanized or metal sprayed structures have been painted, subsequent maintenance will be of the paint system. The paint systems, and their required life to first maintenance of the paint system in very high corrosivity (C5) environment as defined in BS EN ISO 12944 Part 2, to be used for painting galvanized or metal sprayed steelworks shall be : **AMD. 1/2023**

(a) *Paint System I*

To be applied to : parapets, etc.

Life to first maintenance : 7 to 15 years, medium (M) durability as defined in BS EN ISO 12944 Part 1 **AMD. 1/2023**

Pretreatment : two-pack etch primer

Primer : two-pack epoxy primer, 80 μm minimum total dry-film thickness

Finish : two pack epoxy finish coat or polyurethane finish coat, 80 μm minimum total dry-film thickness

(b) *Paint System II*

To be applied to : structural steelworks

Life to first maintenance : 15 to 25 years, high (H) durability as defined in BS EN ISO 12944 Part 1 AMD. 1/2023

Pretreatment : two-pack etch primer

Primer : two-pack epoxy zinc phosphate primer, 80 μm minimum total dry-film thickness

Undercoat : two-pack micaceous iron oxide epoxy undercoat, 140 μm minimum total dry-film thickness

Finish : two-pack polyurethane finish coat, 100 μm minimum total dry-film thickness

(c) *Paint System III*

To be applied to : metal sprayed surfaces

Lift to first maintenance : 15 to 25 years, high (H) durability as defined in BS EN 1SO 12944 Part 1 AMD. 1/2023

Pretreatment : two-pack zinc tetroxychromate polyvinyl butyral pretreatment

Sealer : two-pack epoxy sealer applied by brush until absorption is complete

Primer : two-pack epoxy zinc phosphate primer, 80 μm minimum total dry-film thickness

Undercoat : two-pack micaceous iron oxide epoxy undercoat, 140 μm total minimum dry-film thickness

Finish : two-pack polyurethane finish coat, 100 μm minimum total dry-film thickness

(6) The aforesaid guidelines shall not be applicable to exceptionally massive steelwork, such as the steel deck of the Tsing Ma Bridge, Ting Kau Bridge, etc., where special corrosive protection system shall be considered with regard to the particular project requirements.

16.5 INCORPORATION OF UTILITY INSTALLATIONS IN HIGHWAY STRUCTURES

(1) In general no utility installations other than road lighting, emergency telephones and traffic surveillance equipment will be permitted on highway structures except in cases where there is no other viable routing available. Where other arrangements for a utility line to span an obstruction are not viable nor reasonably practical, the Highways Department may consider the accommodation of such line in a highway structure if the AMD. 1/2018

proposed accommodation will not materially affect the structure, the safe operation of traffic, the efficiency of maintenance and the overall appearance.

(2) The prior approval of Chief Highway Engineer/Bridges and Structures and the maintenance authority shall be sought on any proposal to accommodate utility installations other than road lighting, emergency telephones and traffic surveillance equipment in highway structures. The need for accommodating utility installations should be confirmed at an early stage in the design to allow the designer to make adequate and appropriate provision having due regard to the appearance and functions of the structure. The following guides are established for making provision for accommodation of utilities in highway structures :

(a) The utility lines or installations shall be accommodated in a purpose built trough accessible from the footway or verge, rather than fixed to the sides or underside of the structure using steel brackets. Funding for any additional costs for the provision of the trough is outside the ambit of this Manual and will be dealt with separately.

(b) Encasing utility installations inside the structural elements of the structure including any internal voids is not permitted.

(c) Unless it can be demonstrated that the risks associated with gas main installation are mitigated to an acceptable level, no gas main shall be accommodated in a highway structure. **AMD. 1/2018**

(d) The covers, or covers and frames, for the troughs shall fit closely together to prevent lateral movement or rocking of the covers under traffic. The gap between covers, or covers and frames, shall not exceed 3 mm when assembled in continuous lengths.

(e) Where possible the space under footways and verges should be designed to permit the installation of small utilities at a later date.

16.6 MATERIALS FOR HOLDING DOWN AND FIXING ARRANGEMENTS ON HIGHWAY STRUCTURES

(1) The holding down and fixing arrangements of all sign gantries, noise barriers and the like, and all other fixtures on highways structures shall be fabricated from austenitic stainless steel. In detailing the holding down and fixing arrangements, necessary measures must be provided to prevent galvanic corrosion arising from bi-metallic contact. Stainless steel materials shall comply with Section 18 of the General Specification for Civil Engineering Works, except that Grade 1.4401 shall be replaced by Grade 1.4436 and stainless steel tube shall be Grade 1.4436.

(2) Galvanized mild steel fixing arrangements may only be used for internal fixtures.

16.7 RUNNING SURFACES OF BRIDGE DECKS

(1) To achieve better riding quality and to allow greater flexibility in maintaining the running surfaces of highway bridge decks, the bridge deck surface shall be designed to be finished with bituminous materials in accordance with Highways Department Guidelines No. HQ/GN/25 "Running Surfaces of Bridge Decks". Due consideration shall also be given to the Guidance Notes No. RD/GN/033 "Guidance Notes on the Use of Waterproofing Membranes on Concrete Bridge Decks" published by the Highways Department in designing the bituminous surfacing. **AMD. 1/2023**

(2) A concrete running surface shall only be considered for sections where a short structure is located within a length of rigid carriageway.