

STRUCTURES DESIGN MANUAL FOR HIGHWAYS AND RAILWAYS

2013 Edition

AMENDMENT NO. 1/2026

September 2026

Highways Department

The Government of the Hong Kong
Special Administrative Region



STRUCTURES DESIGN MANUAL FOR HIGHWAYS AND RAILWAYS

2013 Edition

AMENDMENT NO. 1/2026

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 2026, Highways Department reviewed the requirements for wind actions and peak ground accelerations for design. Following the review, amendments to Chapters 3 and 4, as well as Appendices A, B, C, D and E of the SDM, are proposed.

As an environmentally friendly measure, amendments to the SDM will no longer be prepared in form of replacement pages. Continuously updated version of the SDM is available in the departmental website for viewing and downloading.

AMENDMENT DETAILS

The following amendments are made :-

1. CHAPTER 3 ACTIONS

3.2 COMBINATIONS OF ACTIONS

(a) Table 3.1 - Values of ψ Factors for Road Bridges

Replace ψ_0 factor value of “0.75” with “0.75^c” for traffic load components TS and UDL of gr1a.

Add the following after explanatory notes “b”:

^c For ULS wind leading combination, a combined factor $\gamma_Q\psi_0 = 0.30$ shall be applied directly to the accompanying traffic load components TS and UDL of gr1a. For SLS wind leading combination, the value of ψ_0 applied to TS and UDL of gr1a shall be taken as 0.22.

(b) Table 3.3 - Design Values of Actions (EQU) (Set A)

Replace first three bullet points under “Wind actions” as below:

- | | | | |
|---|---|-------------------|----------------------|
| - | Wind velocity pressure derived by Clause 3.4.2(1) | $\gamma_Q = 1.55$ | (0 where favourable) |
| - | Wind velocity pressure derived by Clauses 3.4.2(2) and 3.4.3 | $\gamma_Q = 1.60$ | (0 where favourable) |
| - | Wind actions including aerodynamic effects, where turbulent gust response is considered without vortex excitation | $\gamma_Q = 1.60$ | (0 where favourable) |

(c) Table 3.4 - Design Values of Actions (STR/GEO) (Set B)

Replace first three bullet points under “Wind actions” as below:

- | | | | |
|---|---|-------------------|----------------------|
| - | Wind velocity pressure derived by Clause 3.4.2(1) | $\gamma_Q = 1.55$ | (0 where favourable) |
| - | Wind velocity pressure derived by Clauses 3.4.2(2) and 3.4.3 | $\gamma_Q = 1.60$ | (0 where favourable) |
| - | Wind actions including aerodynamic effects, where turbulent gust response is considered without vortex excitation | $\gamma_Q = 1.60$ | (0 where favourable) |

(d) Table 3.5 – Design Values of Actions (STR/GEO) (Set C)

Replace first three bullet points under “Wind actions” as below:

- | | | | |
|---|---|-------------------|----------------------|
| - | Wind velocity pressure derived by Clause 3.4.2(1) | $\gamma_Q = 1.30$ | (0 where favourable) |
| - | Wind velocity pressure derived by Clauses 3.4.2(2) and 3.4.3 | $\gamma_Q = 1.40$ | (0 where favourable) |
| - | Wind actions including aerodynamic effects, where turbulent gust response is considered without vortex excitation | $\gamma_Q = 1.40$ | (0 where favourable) |

3.4 WIND ACTIONS

(e) Clause 3.4.1 General

Replace sub-clause (3) with the following:

- (3) Waglan Island, which is exposed to southeasterly winds with a long fetch over open sea, is considered as an exposed site to provide the worst wind loading in Hong Kong. Studies show that since Hong Kong is relatively small in climatic terms, the design wind velocity and pressure derived based on the wind data collected from this site are suitable for the design of conventional bridge structures with a degree of conservatism adopted to provide sufficient levels of safety at all sites in Hong Kong. The basic hourly wind velocity for Waglan Island is taken as 49.5 m/s at 70m height for a 120-year return period at an exposed location to derive wind velocity pressure in this Manual.

Delete Table 3.6 in its entirety.

Replace sub-clause (4) with the following:

- (4) The maximum peak wind velocity is related to the peak velocity pressure by the expression:

$$q_p = \frac{1}{2} \rho v_d^2 = 613 \times 10^{-6} v_d^2$$

where q_p = peak velocity pressure (kN/m²)

v_d = maximum peak wind velocity (m/s)

ρ = air density (taken as 1.226 kg/m³).

Replace the numbering of sub-clause “(4)” with “(5)”

Replace sub-clauses (5) and (6) with the following:

- (6) For bridges need not be designed with a “dynamic response procedure”, the procedure specified in Clause 3.4.3 for determining peak velocity pressure shall be followed.
- (7) For the design of walkway covers, sign gantries and noise barriers/enclosures, the procedure given in Clause 3.4.2 shall be followed.

Replace the numbering of sub-clauses “(7)” with “(8)” and “(8)” with “(9)”.

(f) Clause 3.4.2 Simplified Procedure for Determining Peak Velocity Pressure

Delete Tables 3.7 and 3.8a in their entirety.

Replace this clause with the following:

3.4.2 Peak Velocity Pressure for Structures with Design Working Life of 50 Years

- (1) For structures with design working life of 50 years as specified in Clause 2.2(2), i.e. walkway covers, sign gantries and noise barriers/enclosures which are supported at-grade, the peak velocity pressure q_p shall be obtained from Table 3.8.

Table 3.8 – Peak Velocity Pressure for Walkway Covers, Sign Gantries and Noise Barriers/Enclosures

Height above Ground Level (m)	Peak Velocity Pressure q_p (kN/m ²)
≤ 4	1.54
> 4 – 8	1.72
>8 – 12	1.84
>12 – 16	1.92

Where the structure is located at an orography significant site as defined in 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, the peak velocity pressure q_p obtained from Table 3.8 shall be multiplied by a topographical factor $(c_o(z))^2$ in accordance with Annex A.3 of BS EN 1991-1-4 and Clause NA.2.17 of the UK NA to BS EN 1991-1-4.

For height other than the range given in Table 3.8, the designer shall consult the Chief Highways Engineer/Bridges and Structures for advice.

- (2) For the design of sign gantries or noise barriers/enclosures mounted on bridges, the peak velocity pressure shall be obtained according to the provisions given in Clause 3.4.3. A pressure multiplying factor of 0.83 shall be applied for conversion from 120-year return period to 50-year return period. However, for the design of bridges supporting the sign gantries or noise barriers/enclosures, the peak velocity pressure acting on the sign gantries/noise barriers/enclosures and the bridges shall be obtained according to the provisions given in Clause 3.4.3 without reduction.
- (3) For the design of sign gantries or noise barriers/enclosures mounted on bridges, the loaded length for sign gantries or noise barriers/enclosures themselves shall be the length that the applied wind load would create the most adverse impact on these structures, as considered appropriate by the designer, which is irrespective of the bridge span underneath. However, for the design of bridges supporting the sign gantries or noise barriers/enclosures, the loaded length of sign gantries or noise barriers/enclosures shall be taken to be the same as the loaded length of the bridges.

(g) Clause 3.4.3 Full Procedure for Determining Peak Velocity Pressure

Replace the clause title with “Peak Velocity Pressure for Structures with Design Working Life of 120 Years”

Delete the words “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.” in the paragraph under clause 3.4.3.

Replace clause 3.4.3.1 with the following:

3.4.3.1 Peak Velocity Pressure for Wind Leading Combinations

The peak velocity pressure $q_p(z)$ and the hourly mean velocity pressure $q'(z)$ shall be obtained from Table 3.9 appropriate to the height of the bridge and loaded length under consideration. The values in Table 3.9 have taken into account climate change factors. For the bridge heights and horizontal wind loaded lengths falling between those given in Table 3.9, the peak velocity pressure $q_p(z)$ and the hourly mean velocity pressure $q'(z)$ may be obtained by linear interpolation.

Replace Table 3.9 with the following:

Table 3.9 – Peak Velocity Pressure $q_p(z)$ and Hourly Mean Velocity Pressure $q'(z)$

Height z above ground level (m)	Peak Velocity Pressure $q_p(z)$ (kN/m ²) (including climate change factors) appropriate to horizontal wind loaded lengths (m)											Hourly Mean Velocity Pressure $q'(z)$ (kN/m ²)
	20	40	60	80	100	200	400	600	800	1000	2000	
10	2.35	2.28	2.24	2.21	2.18	2.09	1.99	1.93	1.89	1.86	1.78	1.20
15	2.55	2.48	2.44	2.41	2.38	2.28	2.18	2.12	2.08	2.05	1.96	1.32
20	2.69	2.63	2.59	2.55	2.52	2.43	2.32	2.26	2.22	2.19	2.09	1.41
30	2.90	2.84	2.80	2.77	2.74	2.64	2.53	2.47	2.42	2.39	2.28	1.54
40	3.06	3.00	2.96	2.92	2.89	2.79	2.68	2.62	2.57	2.53	2.43	1.64
50	3.18	3.12	3.08	3.05	3.02	2.92	2.81	2.74	2.69	2.65	2.54	1.72
60	3.28	3.23	3.18	3.15	3.12	3.02	2.91	2.84	2.79	2.75	2.64	1.78
70	3.37	3.31	3.27	3.24	3.21	3.11	2.99	2.92	2.87	2.84	2.72	1.84
80	3.45	3.39	3.35	3.31	3.28	3.18	3.07	3.00	2.95	2.91	2.79	1.89
90	3.51	3.46	3.42	3.38	3.35	3.25	3.14	3.07	3.02	2.98	2.86	1.93
100	3.58	3.52	3.48	3.44	3.42	3.32	3.20	3.13	3.08	3.04	2.91	1.97
150	3.82	3.76	3.72	3.69	3.66	3.56	3.44	3.37	3.31	3.27	3.14	2.13
200	3.99	3.94	3.90	3.86	3.84	3.74	3.62	3.54	3.49	3.45	3.31	2.24

Notes:

(1)

For locations which are less exposed, the values $q_p(z)$ and $q'(z)$ given above may be factored according to the degree of exposure as follows :

Degree of exposure	Description	Factor of $q_p(z)$ and $q'(z)$
Sheltered	Sheltered by surrounding buildings and /or topography	0.8
Normal	Normal exposure	0.9
Open Sea	Exposed to north-easterly or south easterly winds across open sea	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 $q_p(z)$ 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 $q_p(z)$ and $q'(z)$ 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 $q_p(z)$ and $q'(z)$ shall be determined based on Annex A.3 of BS EN 1991-1-4.

(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 $q_p(z)$ shall be derived from the centroid of each unit.

Replace Table 3.10 with the following:

Table 3.10 – Velocity Gust Factor (including Multiplication of Velocity Size Factor), G_v

Height z above ground level (m)	Velocity Gust Factor (including multiplication of Velocity Size Factor), G_v appropriate to horizontal wind loaded lengths (m)										
	20	40	60	80	100	200	400	600	800	1000	2000
10	1.40	1.38	1.37	1.36	1.35	1.32	1.29	1.27	1.26	1.25	1.22
15	1.39	1.37	1.36	1.35	1.34	1.32	1.29	1.27	1.26	1.25	1.22
20	1.38	1.37	1.36	1.35	1.34	1.31	1.28	1.27	1.26	1.25	1.22
30	1.37	1.36	1.35	1.34	1.33	1.31	1.28	1.27	1.25	1.24	1.22
40	1.37	1.35	1.34	1.34	1.33	1.31	1.28	1.26	1.25	1.24	1.22
50	1.36	1.35	1.34	1.33	1.33	1.3	1.28	1.26	1.25	1.24	1.22
60	1.36	1.35	1.34	1.33	1.32	1.3	1.28	1.26	1.25	1.24	1.22
70	1.35	1.34	1.33	1.33	1.32	1.3	1.28	1.26	1.25	1.24	1.22
80	1.35	1.34	1.33	1.33	1.32	1.3	1.28	1.26	1.25	1.24	1.22
90	1.35	1.34	1.33	1.32	1.32	1.3	1.28	1.26	1.25	1.24	1.22
100	1.35	1.34	1.33	1.32	1.32	1.3	1.27	1.26	1.25	1.24	1.22
150	1.34	1.33	1.32	1.32	1.31	1.29	1.27	1.26	1.25	1.24	1.22
200	1.33	1.33	1.32	1.31	1.31	1.29	1.27	1.26	1.25	1.24	1.22

Note: Velocity gust factor (including multiplication of velocity size factor) is defined as the ratio between the peak wind velocity and the hourly mean wind velocity at a specified height and horizontal wind loaded length.

Delete clause 3.4.3.6 in its entirety.

(h) Clause 3.4.4 Dynamic Response Procedure

Replace clauses 3.4.4.1, 3.4.4.2 and 3.4.4.3 with the following:

3.4.4.1 General Substitutions

The site mean wind velocity $v_m(z)$ adopted in Clause NA.2.49 of the UK NA to BS EN 1991-1-4 and PD 6688-1-4 Annex A shall be replaced by the values for use in Hong Kong, which shall be taken as :

$$v_m(z) = v_b \cdot c_r(z) \cdot c_o(z)$$

where v_b is the basic hourly mean wind velocity at 10 m height above ground and shall be taken as 44.2 m/s for all sites in Hong Kong. This has taken climate change factors into account and is appropriate to a 120-year return period.

$c_r(z)$ is the roughness factor, which shall be taken as:

$$c_r(z) = 0.123 \times \ln\left(\frac{z}{0.003}\right)$$

$c_o(z)$ is the topographical factor in accordance with Annex A.3 of BS EN 1991-1-4 and Clause NA.2.17 of the UK NA to BS EN 1991-1-4.

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 :

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

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

$$v_d(z) = v_b \cdot G_v \cdot c_o(z)$$

$v_m(z)$, v_b and $c_o(z)$ are defined in Clause 3.4.4.1

G_v is the velocity gust factor (including multiplication of velocity size factor) given in Table 3.10 (for bridge heights and loaded lengths falling between those given in Table 3.10, the velocity gust factor shall be obtained by linear interpolation)

K_{1U} is the factor to cover the uncertainty of prediction of wind velocity and shall be taken as 1.05

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 :

$$v_{WO} = K_{1U} K_{1A} \frac{[v_m(z) + 2v_d(z)]}{3} \quad (\text{for section-model testing})$$

$$v_{WE} = K_{1U} K_{1A} \frac{[v_m(z) + v_d(z)]}{2} \quad (\text{for full-model testing})$$

$$v_{w\alpha} = K_{1U} K_{1A} v_m(z) \quad (\text{for wind inclined to the horizontal by an angle } \alpha \text{ in degrees})$$

where $v_m(z)$ 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)

$$\text{Angle of wind inclination . } \alpha = \bar{\alpha} \pm 7[G_v - 1]$$

2. CHAPTER 4 DESIGN OF EARTHQUAKE RESISTANCE

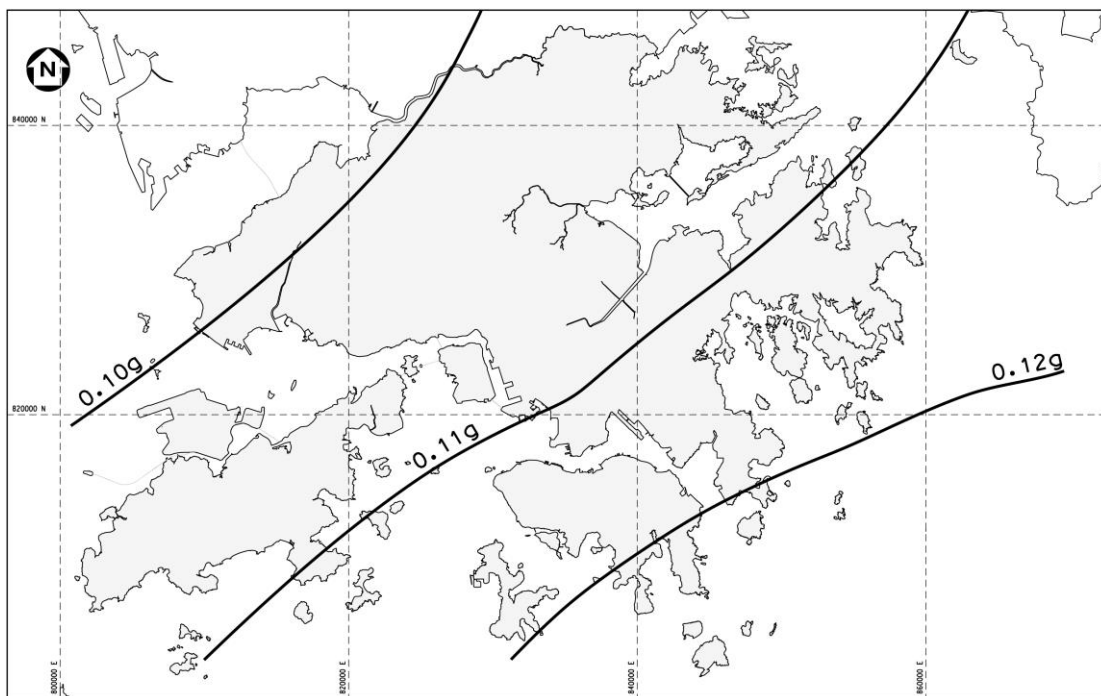
(i) 4.3 REFERENCE PEAK GROUND ACCELERATION

Replace the paragraph under this clause with the following:

In the absence of project specific assessment, the reference peak ground acceleration on type A ground, a_{gR} , that corresponds to the reference return period, T_{NCR} , of 475 years given in Figure 4.1 shall be adopted. The a_{gR} values for sites located between the contour lines in Figure 4.1 shall be obtained by interpolation. The recommendation given in Table NA.1 of the UK NA to BS EN 1998-1 for Clauses 3.2.1(1), (2) and (3) of BS EN 1998-1 which makes reference to the seismic maps in PD 6698 shall not be followed.

Add the following new figure after clause 4.4:

Figure 4.1 – Contour Map for Reference Peak Ground Acceleration on Type A Ground (a_{gR}) Corresponding to Reference Return Period (T_{NCR}) of 475 Years (10% Probability of Being Exceeded in the Next 50 Years)



3. APPENDIX A LIST OF EUROCODES, UK NATIONAL ANNEXES AND PUBLISHED DOCUMENTS

(j) A.2 EUROCODES

Replace “BS EN 1990:2002+A1:2005 Incorporating Corrigenda December 2009”

with “BS EN 1990:2002+A1:2005 Incorporating Corrigenda December 2008” for “Eurocode : Basis of Structural Design”.

Replace “BS EN 1991-1-4:2005+A1:2012” with “BS EN 1991-1-4:2005+A1:2010” for “Eurocode 1: Actions on Structures –Part 1-4: General actions – Wind actions”

4. APPENDIX B EXAMPLES OF COMBINATIONS OF ACTIONS FOR TYPICAL ROAD BRIDGES

(k) B.1 INTRODUCTION

Replace “Clauses 3.4.2.2(1)” with “Clauses 3.4.3.3(1)” in the explanatory note at the end of the clause.

(l) Table B.1 Examples of ULS Combinations of Actions for Typical Road Bridges

Replace “Wind Leading” with “Wind Leading⁷” for the eighth combination in table top.

Replace combined factor $\gamma_{\text{sup}}\psi$ value of “1.01” with “0.30” for traffic load components TS and UDL of gr1a under wind leading combination.

Replace factor γ_{sup} value of “1.55 or 2.10” with “1.60” for load component wind.

Replace combined factor $\gamma_{\text{sup}}\psi$ value of “0.78 or 1.05” with “0.80” for load component wind under traffic leading + wind combination.

Replace combined factor $\gamma_{\text{sup}}\psi$ value of “1.55 or 2.10” with “1.60” for load component wind under wind leading combination.

Replace Clauses “3.4.2.2(1) and Clause 3.4.3.3(1)” with “Clause 3.4.3.3(1)” in table explanatory note 6.

Add the following new explanatory note:

7. For wind leading combination, a combined factor $\gamma_{\text{Q}}\psi_0 = 0.30$ shall be applied directly to the accompanying traffic load components TS and UDL of gr1a.

Replace the numbering of explanatory note “7” with “8”.

(m) Table B.2 Examples of Accidental and Seismic Combinations of Actions for Typical Road Bridges

Replace Clauses “3.4.2.2(1) and Clause 3.4.3.3(1)” with “Clause 3.4.3.3(1)” in table explanatory note 6.

(n) Table B.3 Examples of SLS Combinations of Actions for Typical Road Bridges

Replace “Wind Leading” with “Wind Leading¹⁰” for combinations characteristic and tensile stress verification for prestressed concrete members in table top.

Replace factor ψ value of “0.75” with “0.22” for traffic load components TS and UDL of gr1a under characteristic wind leading combination.

Replace factor ψ value of “0.75” with “0.22” for traffic load components TS and UDL of gr1a for tensile stress verification for prestressed concrete members wind leading combination.

Replace “Clauses 3.4.2.2(1) and Clause 3.4.3.3(1)” with “Clause 3.4.3.3(1)” in table explanatory note 9.

Add the following new explanatory note:

10. For wind leading combination, the value of ψ_0 applied to TS and UDL of gr1a shall be taken as 0.22.

Replace the numbering of explanatory note “10” with “11”.

5. APPENDIX C BACKGROUND TO THE WIND ACTION PROVISIONS FOR HIGHWAY STRUCTURES AND RAILWAY BRIDGES

(o) Clause C.3 WIND LOADING PROVISIONS

Replace the second paragraph with the following:

For bridges that are inherently more flexible, such as cable-stayed or suspension bridges, due consideration must be taken in correctly assessing the dynamic effects of such light and flexible structures.

(p) C.4 BACKGROUND TO CLAUSE 3.4

Replace the first three paragraphs with the following:

This Manual provides procedures for the determination of quasi-static wind loading on highway and railway structures.

Clause 3.4.2 provides peak velocity pressures which are applicable for structures with design working life of 50 years as specified in Clause 2.2(2). This approach is changed in Clause 3.4.3 for structures with design working life of 120 years, where the concept of loaded length is introduced, i.e. the intensity of the loading is a function of the length of loaded part.

Add the following after the fifth paragraph:

The wind loading adopted in this Manual is based on the provisions given in BS EN 1991-1-4 and the corresponding UK NA and PDs, with some parameters replaced by the provisions in this Manual to suit Hong Kong conditions as listed below:

Item	Provisions replaced by this Manual
Hourly Mean Wind Velocity, $v_m(z)$	$v_m(z) = v_b \cdot c_r(z) \cdot c_o(z)$ where v_b is the basic hourly mean wind velocity at 10 m above ground for all sites in Hong Kong based on Waglan Island data of approximate 49.5 m/s at 70 m height, including a climate change factor of 1.106 (Note 1) and appropriate to a 120-year return period: $v_b = \frac{49.5 \times \ln\left(\frac{10}{0.003}\right)}{\ln\left(\frac{70}{0.003}\right)} \times 1.106 = 44.2 \text{ m/s}$ $c_r(z)$ is the roughness factor: $c_r(z) = \frac{0.19 \times \left(\frac{0.003}{0.05}\right)^{0.07}}{\ln\left(\frac{10}{0.003}\right) \times 0.19 \times \left(\frac{0.003}{0.05}\right)^{0.07}} \times \ln\left(\frac{z}{0.003}\right)$ $= 0.123 \times \ln\left(\frac{z}{0.003}\right)$ $c_o(z)$ is the topographical/orographical factor in accordance with Annex A.3 of BS EN 1991-1-4 and Clause NA.2.17 of the UK NA to BS EN 1991-1-4. The following probability factor c_{prob} given in Expression (4.2) in Clause 4.2(2) Note 4 of BS EN 1991-1-4 may be adopted for conversion of wind velocity to a different probability p for an annual exceedance (i.e. with a different return period): $c_{\text{prob}} = \left(\frac{1 - K \cdot \ln(-\ln(1-p))}{1 - K \cdot \ln(-\ln(0.98))} \right)^n$ For conversion from 120-year return period to 50-year return period, the probability factor c_{prob} may be taken as 0.91. (Note 1: 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 recommended that the worst-case scenario, SSP5-8.5, be adopted for the design of highway structures to ensure their resilience and safety under extreme weather conditions. Therefore, a climate change velocity

	multiplying factor of 1.106 shall be taken.)
Velocity Gust Factor, G_v	Velocity Gust Factor, $G_v = \frac{\text{Peak Velocity}}{\text{Hourly Mean Velocity}}$ $G_v(z) = 1.06 \times \sqrt{1 + 7 \times I_v(z)}$ where $I_v(z)$ is the turbulence intensity: $I_v(z) = \frac{1}{\ln \left(\frac{z}{0.003} \right)}$
Velocity Size Factor	Velocity Size Factor = $\sqrt{\text{Pressure Size Factor}}$ $= \sqrt{\frac{1 + 7 \times I_v(z) \times \sqrt{B^2}}{1 + 7 \times I_v(z)}}$ where $B^2 = \frac{1}{1 + 0.9 \times \left(\frac{L}{300 \times (z/200)^{0.38}} \right)^{0.63}}$ L is the loaded length.

(q) C.4.2 Velocity Pressure for Traffic Leading Combinations

Delete “of 35/m/s” in the second paragraph.

(r) C.4.4 Dynamic Response Procedure

Replace the final sentence of the first paragraph with the following:

The designer is recommended to undertake, at least, the following assessments :

(s) C.4.6 ULS Partial Factors

Replace “exceeds” with “is” in the final sentence of the second paragraph.

Replace the text from the third paragraph through to the final paragraph with the following:

The wind load factor of 1.6 is derived from extreme value analysis based on Type 1 or Gumbel distributions on the square of the maximum wind speed (V^2), with a notional return period of 2400 years for ultimate limit state (ULS). This has included the additional γ_{f3} factor of 1.1 used for determining design load effects in previous SDMHR.

Care should be taken in designing ‘landmark’ structures that may be more prone to

wind load effects. If the designer is in doubt over the loading requirements for a structure because of a perceived need for a higher structural reliability, the designer should consult the Chief Highways Engineer/Bridges and Structures for advice.

6. APPENDIX D BACKGROUND ON AERODYNAMIC EFFECTS ON BRIDGES

(t) D.1 INTRODUCTION

Replace “There are three principal” with “The following are common” in the fourth paragraph.

(u) D.2 DIVERGENT RESPONSE

Delete “Divergent responses include both galloping and flutter.” at the beginning of the second paragraph.

Replace “in a return period of 120 years” with “within the design life of the bridge” in the final paragraph.

(v) D.4 TURBULENCE RESPONSE

Delete “The cross-wind forces are generated by changes in the angle of the incident wind from horizontal due to the presence of large gusts.” in the third paragraph.

Delete the fourth paragraph in its entirety.

Delete “(but close to the ground the vertical component becomes less)” in the ninth paragraph.

(w) D.5 COMBINATION OF TURBULENT RESPONSES WITH OTHER AERODYNAMIC LOAD EFFECTS

Delete this clause in its entirety.

7. APPENDIX E GUIDANCE FOR WIND TUNNEL TESTING

(x) E.1 INTRODUCTION

Replace “Wind tunnel testing is expensive and.....to the commencement of testing.” with “The designer needs to know what tests are required to justify the design and what information should be collected for developing the testing proposal.”

Delete the second paragraph in its entirety.

(y) E.2 GENERAL REQUIREMENTS FOR WIND TUNNEL TESTING

Replace the first two paragraphs with the following:

There are three basic reasons for undertaking wind tunnel tests. Firstly, the tests may be undertaken to obtain static coefficients for use in the basic static design checks for wind or for input to analysis of turbulence response. In the first instance, these data may be obtained from tests undertaken on other structures or by referring to the relevant design code or literature. However, relevant design code or literature do not cover every eventuality and the coefficients provided have a degree of conservatism built in to ensure they apply to a wide range of structures. Thus, if wind forces contribute substantially to the cost of the structure, benefit may be gained in determining the static coefficients more precisely through wind tunnel tests. Clearly, the cost of the wind tunnel tests must be offset in the savings achieved in the design of the bridge.

Secondly, the tests will allow checks on limited amplitude responses or divergent amplitude responses. Such tests require dynamic models and can also yield either a direct estimation of turbulence response or 'derivative' coefficients, which enable more sophisticated numerical analysis of turbulence response to be carried out. The criteria for establishing the need to undertake these tests are given in PD 6688-1-4 and CD 363.

Delete the fourth and the fifth paragraphs in their entirety.

Add "objectives of the tests as well as the" after "This must depend on the" in the seventh paragraph.

Replace "is well known to those involved in the testing;" with ", as well as the design of the parapet of the Stonecutters Bridge, are well known to those involved in the testing;" in the ninth paragraph.

(z) E.3.1 Determination of Time Average Coefficients

Replace "The size of the model should not be so large as to create significant" with "While the Reynold number (Re) effect, especially for those sections comprising circular section members or other curved surfaces, should be considered, a large size model would cause" in the first paragraph.

Replace "BD 49/01 requires values of less than this and the guidance given therein" with "CD 363 provides guidance on this value and" in the third paragraph.

Delete the second and the fourth paragraphs in their entirety.

(aa) E.3.2 Determination of Overall Wind Loads

Delete this clause in its entirety.

(bb) E.3.3 Section Model Tests to Determine Aerodynamic Stability

Delete the second paragraph in its entirety.

Replace the numbering of clause "E3.3" with "E3.2".

(cc) E.3.4 Aerodynamic Derivatives

Replace the numbering of clause “E3.4” with “E3.3”.

(dd) E.4 AEROELASTIC SIMULATIONS OF BRIDGES

Replace “In such cases the model is built to correctly represent the.....tests must also consistently model the salient characteristics of natural wind at the site.” **with** “In such cases the model is built to correctly represent the dynamic properties and damping of the concerned modes of vibrations and the aerodynamically significant features of the bridge’s geometry. The tests must also consistently model the salient characteristics of natural wind at the site.” **in the first paragraph.**

Delete the second paragraph in its entirety.