

Lattice towers and masts —

**Part 2: Guide to the background and use
of Part 1 “Code of practice for loading”**

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Foreword

This Part of BS 8100 has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee. BS 8100 is a standard combining a code of practice and a guide covering the loading and design of lattice towers of metallic construction.

It comprises the following Parts:

- *Part 1: Code of practice for loading;*
- *Part 2: Guide to the background and use of Part 1 “Code of practice for loading”.*

DD 133 covers the strength assessment of members.

This Part of BS 8100 does not apply to other metallic structures. Other British Standards exist for some of those structures.

It has been assumed in the drafting of this British Standard that the execution of its provisions will be entrusted to appropriately qualified and experienced people.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 170, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

1 Scope

This Part of BS 8100 contains both guidance on the interpretation of and background to the use of BS 8100-1, which is the code of practice covering the loading of lattice towers and masts.

NOTE 1 Where references are made in the text to “Part 1” they refer to BS 8100-1:1986.

Two worked examples are provided to assist designers with the application of Part 1.

NOTE 2 The titles of the publications referred to in this standard are listed on the inside back cover.

2 Definitions

For the purposes of this Part of BS 8100, the definitions given in Part 1 apply.

3 Symbols

The main symbols and subscripts used in this Part of BS 8100 are as given in Part 1. However, due to the nature of this Part of BS 8100, it has not been possible to adopt a unique set of symbols and hence all additional symbols are defined as they appear in the text.

4 Worked examples

Two worked examples are given in Appendix A and Appendix B as guidance to Part 1. They both relate to typical broadcasting towers, but the method of calculation is equally applicable to other types of tower. The examples show how the appropriate clauses of Part 1 may be used in practice, with some guidance on interpretation and simplification, where appropriate. Such guidance may only relate to the examples chosen and should not be taken as being applicable in all situations. Only sufficient calculations have been provided to show the method of calculation required.

Although worked example 1 (see Appendix A) is a smaller tower than worked example 2 (see Appendix B) with fewer aerials and attachments, the two examples should not be considered as simple and complex, respectively. In order to cover as many as possible of the recommendations of Part 1, considerations of interpretation of wind data, hill sites, vortex shedding, serviceability and fatigue have been covered in worked example 1, since they are not relevant to the particular example chosen for worked example 2, which concentrates more thoroughly on the full (general) method of assessment of wind resistance given in 4.3 and 4.4 of Part 1 and the consequent loading considerations of 5.3 of Part 1.

The examples are referenced throughout to the relevant clauses of Part 1 (and this Part of BS 8100) in typical calculation format. As a further aid to the user of Part 1, Table 4.1 is provided listing the references in Appendix A and Appendix B in which calculations (or notes) relating to the clauses of Part 1 can be found.

The numbering of Appendix A and Appendix B is generally aligned with the numbering of Part 1 so that equivalently numbered subclauses cover the same subject matter in order to aid cross-referencing.

5 Background to the use of Part 1

The background to the use of Part 1 is given in Appendix C.

In Appendix C, reference is made to source material and where new procedures have been developed, particularly those concerning safety provisions and gust response of structures, the theoretical background is summarized. Comparison is made between the results of application of the various clauses of Part 1, existing codes and experimental evidence.

The numbering of Appendix C has been aligned with the numbering of Part 1 so that equivalently numbered subclauses cover the same subject matter in order to aid cross-referencing. Additional subclauses have been included in Appendix C covering different areas of the subject matter to those in Part 1, or where additional background information is considered to be useful.

NOTE The numbers in square brackets used throughout Appendix C refer to the bibliographic references given in C.1.3, C.2.8, C.3.8, C.4.10 and C.5.12.

Table 4.1 — References to Part 1 covered by the worked examples

Reference to Part 1			Reference in worked example	
Clause	Figure	Table	Example 1	Example 2
Section 1. General				
1.1 Scope				
1.2 Definitions				
1.3 Symbols				
1.3.1 Main symbols				
1.3.2 Subscripts				
Section 2. Performance requirements				
2.1 Safety and service life			A.2.1	B.2.1
2.1.1 General				
2.1.2 Service life			A.2.1	B.2.1
2.2 Classification of required reliability			A.2.2	B.2.2
2.2.1 General			A.2.2	
2.2.2 Environmental conditions			A.2.2	B.2.2
2.2.3 Economic consequences of usage			A.2.2	B.2.2
2.3 Classification of quality			A.2.3	B.2.3
2.3.1 General				
2.3.2 Class A towers			A.2.3	B.2.3
2.3.3 Class B towers				
2.3.4 Class C towers				
2.4 Safety factors			A.2.2	B.2.4
2.4.1 Wind speed and ice thickness factors	Figure 2.1		A.2.2	B.2.4
2.4.2 Dead load and partial factors	Figure 2.1		A.2.4	B.2.4
2.5 Safety assessment	Figure 2.1		A.6.1	B.2.4, B.6
2.6 Serviceability			A.6.3	
Section 3. Meteorological parameters				
3.1 Wind speed data			A.3.1	B.3.1
3.1.1 General			A.7.2	
3.1.2 Basic wind speed	Figure 3.1		A.3.1, A.7.2	B.3.1
3.1.3 Wind direction factor	Figure 3.2		A.3.1, A.7.2	B.3.1
3.1.4 Terrain roughness factor	Figure 3.3	Table 3.1	A.3.1, A.7.2	B.3.1
3.1.5 Site reference wind speed			A.3.1, A.6.3, A.7.2	B.3.1
3.2 Variation of wind speed with height			A.3.2	B.3.2
3.2.1 Sites on level terrain	Figure 3.3		A.3.2, A.7.2	B.3.1, B.3.2, B.5.1
3.2.2 Sites on hills	Figure 3.4, Figure 3.5		A.6.5, A.7.1	
3.3 Serviceability			A.3.3, A.6.2, A.6.3	B.3.3
3.3.1 Distribution of wind speeds				
3.3.2 Downwind deflections	Figure 3.6		A.6.3	B.5.3
3.3.3 Crosswind deflection (vortex shedding)	Figure 3.7, Figure 3.8		A.6.3, A.6.4	

Table 4.1 — References to Part 1 covered by the worked examples

Reference to Part 1			Reference in worked example		
Clause		Figure	Table	Example 1	Example 2
3.4	Fatigue life assessment	Figure 3.9		A.3.4	B.3.4
3.4.1	In-line vibrations			A.6.4	
3.4.2	Crosswind vibrations			A.6.2, A.6.4	
3.5	Ice loading			A.3.5	B.3.5
3.5.1	General			A.3.5	B.3.5
3.5.2	Basic ice thickness			A.3.5	B.3.5
3.5.3	Reference ice thickness			A.3.5	B.3.5
3.5.4	Ice weight			A.3.5	B.3.5
Section 4. Wind resistance					
4.1	General	Figure 4.1		A.4.1	B.4.1
4.1.1	Method of derivation			A.4.1	B.4.1
4.1.2	Symmetrical towers without ancillaries			A.4.1	B.4.2, B.4.3, B.4.4
4.1.3	Symmetrical towers with limited ancillaries			A.4.1	B.4.3, B.4.4
4.1.4	Other cases	Figure 4.2 Figure 4.3			B.4.4
4.2	Method for symmetrical towers without ancillaries			A.4.1, A.4.3	B.4.2, B.4.5
4.2.1	Calculation of total wind resistance			A.4.1, A.4.3	B.4.2, B.4.3
4.2.2	Overall drag coefficients			A.4.3	B.4.2
4.3	Method for symmetrical towers with limited ancillaries				B.4.5
4.3.1	Constraints for simplified method				B.4.3
4.3.2	Calculation of total wind resistance				B.4.3, B.5.1
4.4	General method for towers containing ancillaries or unsymmetrical towers				B.4.5
4.4.1	Calculation of total wind resistance	Figure 4.4 Figure 4.5			B.4.4
4.4.2	Drag (pressure) coefficients for single frames				B.4.4
4.5	Linear ancillaries		Table 4.1, Table 4.2	A.4.2	B.4.3, B.4.4
4.6	Discrete ancillaries				B.4.4
4.7	Cables				
4.8	Icing			A.3.5	
Section 5. Structural response to wind					
5.1	Procedure			A.5.1	B.5.1
5.1.1	General			A.5.1	B.5.1
5.1.2	Equivalent static method			A.5.1	B.5.1
5.1.3	Spectral analytical method				
5.1.4	Deflections			A.6.3	B.5.3
5.1.5	Vortex-excited vibrations			A.6.2	

Table 4.1 — References to Part 1 covered by the worked examples

Reference to Part 1			Reference in worked example	
Clause	Figure	Table	Example 1	Example 2
5.2 Wind loading for symmetrical towers				
5.2.1 General			A.5.1	B.5.1
5.2.2 Basic gust response factor	Figure 5.1, Figure 5.2 Figure 5.3		A.5.1	B.5.1
5.2.3 Loading for calculating bending moments			A.5.1	B.5.1
5.2.4 Loading for calculating shear forces	Figure 5.4		A.5.1	B.5.1
5.2.5 Loading for calculating deflections			A.6.3	B.5.3
5.2.6 Calculation of wind forces in tower members			A.5.2	
5.3 Wind loading for towers with complex attachments				B.5.1
5.3.1 General				B.4.6, B.5.1
5.3.2 Loading on ancillary items				B.5.1
5.3.3 Loading on cables	Figure 5.5 to Figure 5.7			
5.3.4 Calculation of wind forces in tower members				B.5.2
5.3.5 Loading for calculating deflections				B.5.3
5.4 Spectral analytical method				
5.5 Crosswind response due to vortex excitation			A.6.2	B.3.3, B.3.4
5.5.1 Critical wind speed			A.6.2	
5.5.2 Excitation	Figure 5.8		A.6.2, A.6.3	
Appendix A Measurement and interpretation of wind data	Figure A.1		A.7.2	
Appendix B Terrain parameters			A.7.2	
Appendix C Gradient wind speeds	Figure C.1, Figure C.2		A.7.2	
Appendix D Hill parameters in undulating terrain	Figure D.1		A.7.1	
Appendix E Parameters for spectral analytical methods		Table E.1, Table E.2		
Appendix F Method of assessment for classification of quality		Table F.1, Table F.2	A.2.3	B.2.3
Appendix G Equations used for the production of curves used in the figures				

Appendix A Worked example 1

Reference in Part 1	Worked example 1
	A.1 Site and tower details A.1.1 General The structure is a 30 m high triangular unmanned broadcasting tower with cylindrical aerial shroud on top. Location is East Riding, Yorkshire at 170 m above mean sea level (AMSL), situated in very open flat moorland with very few obstructions (hedges, fences, trees, etc.). A.1.2 Form of structure From preliminary considerations and previous experience, member sizes as shown in Figure A.1 have been adopted.
2.1 2.1.2 2.2, 2.4 2.2.1 2.2.2 2.2.3 Figure 2.1	A.2 Performance requirements A.2.1 Service life The intended service life of the structure is 30 years. A.2.2 Performance requirements and safety factors No specific cost criteria have been provided, and hence safety factors are selected from: a) environment, i.e. potential hazards; b) usage of the tower. For a) the environmental category is “unmanned in open countryside”. For b) the usage is “broadcasting”. From Figure 2.1 of Part 1, it can be seen that b) governs and the governing γ_v is taken as 1.15, i.e. appropriate to the right-hand limit of the governing performance requirement, which in this case has been assumed to have been specified by the client. The corresponding γ_{DL}, and γ_m are then: $\gamma_{DL} = 1.01$ for dead load effects increasing wind effects; or 0.90 for dead load effects decreasing wind effects; $\gamma_m = 1.10$ for quality class A (see note to A.2.3). NOTE 1 If possible, due account should be made of any known future change to the environment. If, for example, a main road was to be situated adjacent to the tower, the governing performance requirement would then be a) environment: “adjacent to main road” giving increased safety factors thus: $\gamma_v = 1.20$; $\gamma_{DL} = 1.05$. However, provided no such criteria need be considered, the required safety factors are: $\gamma_v = 1.15$; $\gamma_{DL} = 1.01$ or 0.90, according to load effect; $\gamma_m = 1.10$ for quality class A These values can be confirmed by inspection of Table C.2.2. NOTE In this example the usage criteria have been used, and the right-hand limit from “broadcasting” adopted. The left-hand limit, i.e. $\gamma_v = 1.07$, would be used if no value had been specified by the client.
(C.2.5 of this part of BS 8100) 2.2.3.3	

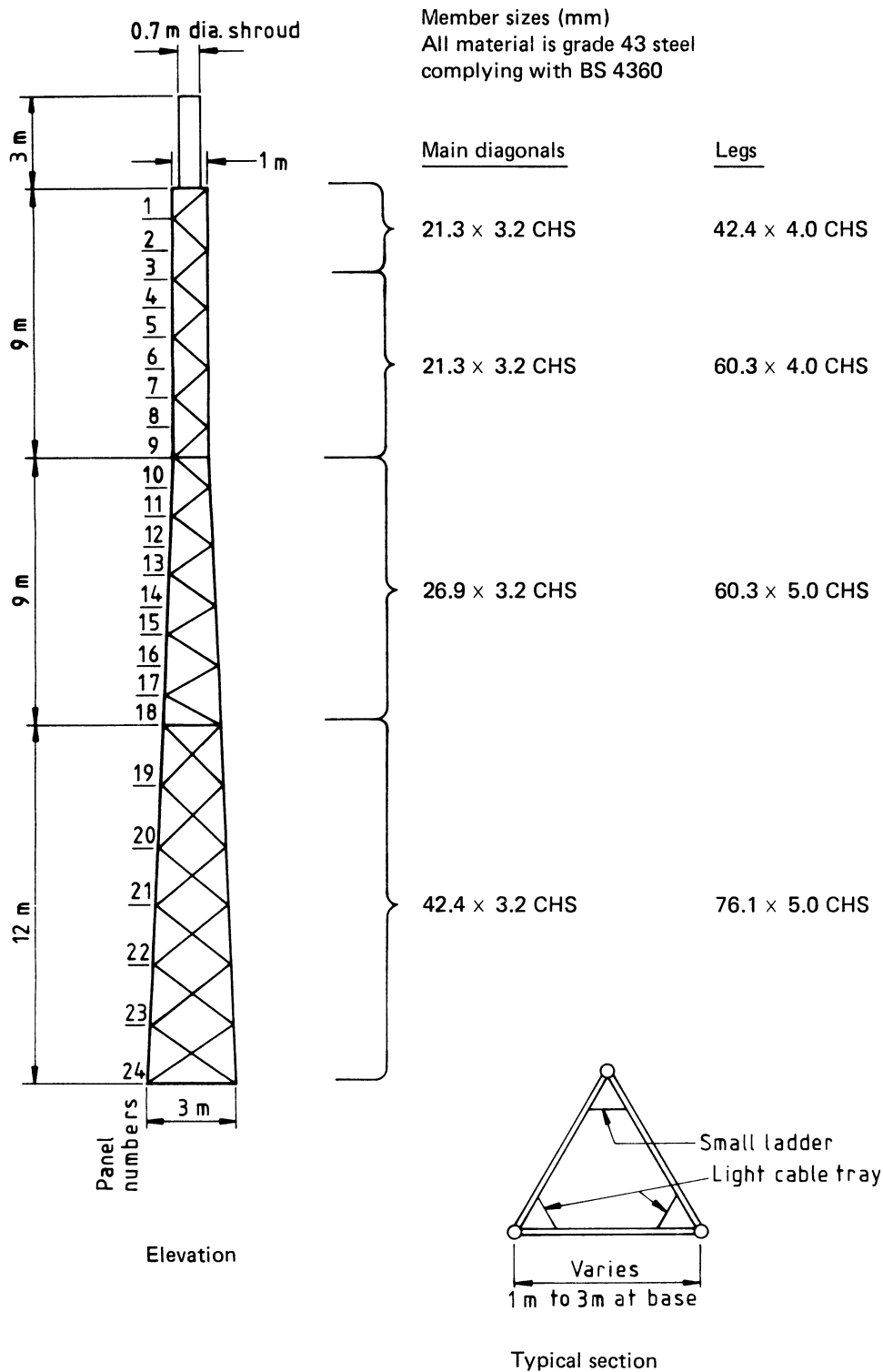


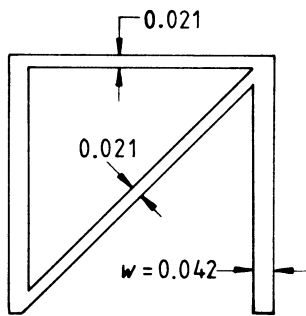
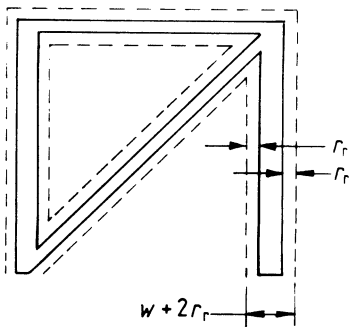
Figure A.1 — Worked example 1. 30 m broadcasting tower with shroud (general arrangement)

Reference in Part 1	Worked example 1																		
2.2.3.2	The consequential cost ratio appropriate to selected criteria can then be evaluated in accordance with 2.2.3.2 of Part 1. Hence for the selected right-hand limit of the “broadcasting” category; $\log_{10}(gi_s)$ is in the range 1.75 to 1.98 For $i_s = 30$ years, this gives g in the range 1.9 to 3.2, which is the ratio of the consequential cost of replacement to the initial cost, and can be reported back to the client, who may require a higher allowance. Alternatively, if a γ_v of 1.20 was chosen in accordance with note 1 for the “adjacent to main road” category this gives: $\log_{10}(gi_s)$ in the range 1.9 to 2.2 Again for $i_s = 30$ years, this gives g in the range 2.6 to 5.3 Hence it can be seen that a full assessment of all possible performance requirements is required before finalizing the appropriate safety factors.																		
2.4																			
2.3	A.2.3 Design, fabrication and erection standards and tower classification The structural design will be in accordance with Part 1 and DD 133, the design and detailing will be independently appraised, the materials, quality control and workmanship will be in accordance with the appropriate British Standard, and the tower will be subject to inspection after fabrication. Routine inspections will be conducted every 2 years after erection. On this basis, the tower may be considered to be a class A tower, as it complies with all the recommendations of 2.3.2 of Part 1. NOTE Alternatively, Appendix F of Part 1 could be used for the assessment of quality classification. This gives the following rating from Table F.1 of Part 1: <table><tr><td>Item 1:</td><td>(b)</td><td>Rating 1</td></tr><tr><td>Item 2:</td><td>(b)</td><td>Rating 1</td></tr><tr><td>Item 3:</td><td>(a)</td><td>Rating 1</td></tr><tr><td>Item 4:</td><td>(a)</td><td>Rating 2</td></tr><tr><td>Item 5</td><td>(a)</td><td>Rating 2</td></tr><tr><td colspan="2"></td><td> Total 7</td></tr></table>	Item 1:	(b)	Rating 1	Item 2:	(b)	Rating 1	Item 3:	(a)	Rating 1	Item 4:	(a)	Rating 2	Item 5	(a)	Rating 2			Total 7
Item 1:	(b)	Rating 1																	
Item 2:	(b)	Rating 1																	
Item 3:	(a)	Rating 1																	
Item 4:	(a)	Rating 2																	
Item 5	(a)	Rating 2																	
		Total 7																	
2.3.2 Appendix F Table F.1																			
Table F.2	From Table F.2 of Part 1, the quality category is therefore class A. It should be noted that if there was no independent design check <i>or</i> there was no regular inspection and maintenance procedure, the rating would drop to 6 (or 5) and the quality category would become class B, in which case γ_v and γ_{DL} would be unchanged, but γ_m would become 1.20)																		
2.4.2.2	A.2.4 Dead loads Dead loads resulting from the self weight of the tower, including the weight of the ladder, cable trays and aerial structure, are calculated in accordance with 2.4.2.2 of Part 1, with appropriate allowances for connections, etc.																		
3.1 3.1.2 Figure 3.1	A.3 Meteorological parameters A.3.1 General a) <i>Basic wind speed</i>. The map wind speed for the site is given as 24 m/s at 10 m AMSL in Figure 3.1 of part 1. Allowance has to be made for the altitude, giving the basic wind speed; $\bar{V}_B = 1.17 \times 24 = 28.1\text{m/s}$																		
3.1.3	b) <i>wind direction factor</i>. Since the tower is on sensibly level ground and uniform terrain [see c)] and is symmetric with regard to wind resistance, no advantage can be gained from the use of Figure 3.2 of part 1. Consequently, the wind direction factor, K_d, has been taken as 1.0 for ice-free conditions, and 0.85 for the combination of wind with ice.																		

Reference in Part 1	Worked example 1										
3.1.4 Figure 3.3 or Table 3.1 (C.3.1.4 of this part of BS 8100)	c) <i>Terrain roughness parameters.</i> The site is an open moorland with very few isolated obstructions. Consequently, from Table 3.1 of Part 1 it may be deemed to be in category II from which the appropriate parameters for the site are as follows: K_R $= 1.10;$ α $= 0.14;$ h_e $= 0.$ NOTE These values are confirmed by inspection of Figure C.3.1										
3.1.5	d) <i>Site reference wind speed, $\bar{V}_r$.</i> The site reference wind speed is given by: $\bar{V}_r$ $= \gamma_v K_d K_R \bar{V}_B$ $= 1.15 \times 1.0 \times 1.1 \times 28.1$ $= \underline{35.5 \text{ m/s}}$ Also the characteristic wind speed is given by: $\bar{V}_k$ $= K_R \bar{V}_B$ $= 1.10 \times 28.1$ $= \underline{30.9 \text{ m/s}}$ $\bar{V}_k$ is required for the serviceability check (see A.3.3).										
3.2 3.2.1	A.3.2 <i>Wind profile</i> The tower is situated on level ground, therefore 3.2.1 of Part 1 is used to obtain the wind profile. NOTE An example of calculating the wind profile for a site on a hill is given in A.7.1. Using $\bar{V}_z$ $= 35.5 \left(\frac{z}{10} \right)^{0.14}$ for $z \geq 10 \text{ m}$ and $\bar{V}_z$ $= 17.75 \left(1 + \frac{z}{10} \right)$ for $z < 10 \text{ m}$ The values of $\bar{V}_z$ given in Table A.3.1 are obtained for typical levels. Table A.3.1 — Mean wind speed, $\bar{V}_z$ <table><tr><td>Panel height, z (m)</td><td>1.0</td><td>12.5</td><td>21.5</td><td>27.5</td></tr><tr><td>$\bar{V}_z$ (m/s)</td><td>19.5</td><td>36.6</td><td>39.5</td><td>40.9</td></tr></table>	Panel height, z (m)	1.0	12.5	21.5	27.5	$\bar{V}_z$ (m/s)	19.5	36.6	39.5	40.9
Panel height, z (m)	1.0	12.5	21.5	27.5							
$\bar{V}_z$ (m/s)	19.5	36.6	39.5	40.9							
3.3	A.3.3 <i>Serviceability</i> Serviceability is dealt with after the derivation of loads and strength (safety) assessment and the appropriate details are given in A.6.3.										
3.4	A.3.4 <i>Fatigue life assessment</i> Fatigue life assessment is dealt with after the derivation of loads and strength (safety) assessment and the appropriate details are given in A.6.4.										

Reference in Part 1	Worked example 1
3.5	A.3.5 Ice loading
3.5.1	a) <i>Wind speed</i>. The site reference speed to be used with ice, $\bar{V}_i$, is given by: $\bar{V}_i = 0.8 \gamma_v K_d K_R \bar{V}_B$ $= 0.8 \times 1.15 \times 0.85 \times 1.1 \times 28.1$ $= 24.2 \text{ m/s} = 0.68 \bar{V}_r$
3.5.2	The variation of $\bar{V}_i$ with height follows that of $\bar{V}_r$, with $\bar{V}_{zi}$ being $0.68 \times \bar{V}_z$, where values of $\bar{V}_z$ are given in Table A.3.1.
3.5.2.1	b) <i>Basic ice thickness</i> 1) In the absence of wind: $r_{Bo} = k_i \left\{ r_o + \left(\frac{a - 200}{25} \right) \right\} \geq k_i r_o$ where $k_i = \left(\frac{2}{3} + \frac{4}{D} \right)$ D varies from 21.3 mm to 76.1 mm; $a = 200$ m (site is at 170 m AMSL); $r_o = 60$ mm.
Figure 3.9 3.5.2.2	2) With wind: $r_{Bw} = k_i \left\{ r_w + \left(\frac{a - 200}{25} \right) \right\} \geq k_i r_w$ where $r_w = 10 \text{ mm}$
Figure 3.9 4.8	There are no gaps of less than 75 mm, and hence there are no significant areas that need to be considered as completely filled with ice.
3.5.3	c) <i>Reference ice thickness</i>. The reference ice thickness is given by: $r_r = \gamma_v K_C r_B$ $= 1.15 \times 1.0 \times r_B$ i.e. $r_{ro} = 69 k_i \text{ mm}$; $r_{rw} = 11.5 k_i \text{ mm}$
3.5.4	This gives design values as shown in Table A.3.2. d) <i>Ice weight</i>. The unit weight of ice is: $\rho_{io} = 5 \text{ kN/m}^3 \text{ for no wind};$ $\rho_{iw} = 9 \text{ kN/m}^3 \text{ for wind and compression};$ $\rho_{iw} = 5 \text{ kN/m}^3 \text{ for wind and uplift}.$ Ice loads are then calculated on the basis of: $I = \pi \rho_i r_r (d + r_r) \text{ kN/m}$ with r_r and member diameter, d, in metres, using $\rho_{iw} = 9 \text{ kN/m}^3$ for I_w. Where required, these values can then be factored by 0.56 for uplift consideration. Values of I_o and I_w are given in Table A.3.2.

Reference in Part 1	Worked example 1								
4.1 4.1.1 Note to 4.1.3 4.1.2 4.2.1 a) 4.2.1 b) 4.2 4.5 Table 4.1	Table A.3.2 — Ice thickness and weight for typical members								
	Member diameter	k_i	Member weight	No wind			With wind		
				r_{Bo}	r_{ro}	I_o	r_{Bw}	r_{rw}	I_w
	mm		kN/m	mm	mm	kN/m	mm	mm	kN/m
	21.3	0.85	0.014	51	59	0.074	9	10.0	0.009
	33.7	0.79	0.024	47	54	0.074	8	9.1	0.011
	76.1	0.72	0.086	43	50	0.099	7	8.3	0.020
	A.4 Wind resistance								
	A.4.1 General								
	For analysis, the tower is divided into 24 panels using leg/bracing intersections. The tower is mounted with a cylindrical shroud whose resistance should be calculated separately. Each lattice panel encompassing one set of primary bracing has been considered for the purposes of calculating wind resistance. For such a simple tower some of these panels may in practice be aggregated, but in this example full panel divisions are retained to demonstrate the calculation procedure more generally.								
The ancillaries consist of two small cable trays and a small ladder.									
The small cable trays and ladder are reasonably symmetrical in each face and can hence be treated as structural members for the purposes of wind resistance, as can the cylindrical shroud, and hence wind resistance can be determined by means of the overall drag coefficients given in 4.2 of Part 1, i.e.:									
$\Sigma R_w = R_T = K_\theta C_N A_F$									
The crosswind resistance ΣR_x for a tower which is symmetric can be considered as the same as the in-line wind resistance, ΣR_w .									
A.4.2 Cylindrical shroud									
The overall resistance for this is derived in accordance with 4.5 of Part 1, using Table 4.1 of Part 1.									
The effective Reynolds number is given by:									
$R_e = \frac{1.5 \bar{V}_z D}{\nu} = \frac{1.5 \times 42 \times 0.7}{1.46 \times 10^{-5}} = 30 \times 10^5$									
This is greater than 10×10^5 and hence for supercritical flow:									
$C_N = 0.7$ (ice free) or 1.0 (iced);									
$K_A = 1.0$;									
$A_A \sin^2 \psi = 0.7 \times 3.0 = 2.1 \text{ m}^2$									
$\Sigma R_W = R_{AW} = C_N K_A A_A \sin^2 \psi$									
$= 0.7 \times 1.0 \times 2.1 = 1.47 \text{ m}^2$ (ice free)									
or									
$= 1.0 \times 1.0 \times 2.1 = 2.10 \text{ m}^2$ (iced)									
The maximum ice-free resistance for subcritical flow will occur when $R_e = 2 \times 10^5$ compatible with a wind speed of about 3 m/s. Clearly this will not be a governing condition.									

Reference in Part 1	Worked example 1
4.2.1	A.4.3 Panels 1 to 24, lattice sections The flow around the circular-section members has to be checked to see which flow regime exists. At the tower top: $\bar{V}_z = 41.3 \text{ m/s}$ Hence minimum D for supercritical flow: $D \geq R_e \nu / 1.5 \bar{V}_z = 94 \text{ mm}$ At a height of 12 m (top of 76 mm section) $\bar{V}_z = 36 \text{ m/s}$: $D \geq 108 \text{ mm for supercritical flow}$ Since all panel members are smaller than these limiting values, all circular-section members may be taken as having subcritical flow, and no reduction can be made for supercritical flow. Resistance without ice  $A_c = 1.0 \times 0.42 \times 2 + (1.0 + 1.414) \times 0.021$ $= 0.134 \text{ m}^2$ $\phi = \frac{1.05 \times 0.134}{1.0 \times 1.0}$ $= 0.14$ Resistance with ice  $A_c = 1.0 \times (0.042 + 2 \times 0.009) \times 2 + (1.0 + 1.414) \times (0.021 + 2 \times 0.01)$ $= 0.218 \text{ m}^2$ $\phi = \frac{1.05 \times 0.218}{1.0 \times 1.0}$ $= 0.23$ Panels 4 to 9 $A_c = 1.0 \times 0.060 \times 2 + 1.414 \times 0.0213$ $= 0.150 \text{ m}^2$ $\phi = \frac{1.05 \times 0.151}{1.0 \times 1.0}$ $= 0.16$ $A_c = 1.0 \times (0.060 + 2 \times 0.009) \times 2 + 1.414 (0.021 + 2 \times 0.010)$ $= 0.214 \text{ m}^2$ $\phi = \frac{1.05 \times 0.215}{1.0 \times 1.0}$ $= 0.23$ This process is repeated in turn for each panel. Typical results have been tabulated in Table A.4.1. The ancillaries make up approximately 5 % of the tower area in this case, and are all flat-sided members. Since they are reasonably identical on each face, they may be treated as structural members and the tower resistance calculated in accordance with 4.2 of Part 1.
4.2	

Reference in Part 1	Worked example 1																																																																																																																																																																																																								
4.2.1 4.2.2 Figure 4.2 Figure 4.3	Values of A_c, A_f and A_s ($= A_c + A_f$) are given in Table A.4.1, and the resistances of each panel are set out in Table A.4.2 derived from: $R_w = K_\theta C_N A_s$ Where $C_N = \frac{A_f}{A_s} C_{Nf} + \frac{A_c}{A_s} C_{Nc} + \frac{A_{c'}}{A_s} C_{Nc'};$ $K_\theta = 1.0$ from Figure 4.2 of Part 1; C_{Nf}, C_{Nc} from Figure 4.3 b) of Part 1. Table A.4.1 — Typical projected areas of tower panels <table><tr><th rowspan="2">Panel no.</th><th rowspan="2">Member</th><th rowspan="2">Length</th><th colspan="2">Width</th><th rowspan="2">Panel no.</th><th colspan="3">Projected area</th></tr><tr><th>Uniced</th><th>Iced</th><th>Type</th><th>Uniced</th><th>Iced</th></tr><tr><td rowspan="4">3</td><td>Leg</td><td>1.00</td><td>0.042</td><td>0.060</td><td rowspan="4">3</td><td>A_c</td><td>0.114</td><td>0.178</td></tr><tr><td>Diag.</td><td>1.41</td><td>0.021</td><td>0.041</td><td>A_f</td><td>0.006</td><td>0.009</td></tr><tr><td>Horiz.</td><td>NA</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr><tr><td></td><td></td><td></td><td></td><td>A_s</td><td>0.120</td><td>0.187</td></tr><tr><td rowspan="4">9</td><td>Leg</td><td>1.00</td><td>0.060</td><td>0.078</td><td rowspan="4">9</td><td>A_c</td><td>0.150</td><td>0.214</td></tr><tr><td>Diag.</td><td>1.41</td><td>0.021</td><td>0.041</td><td>A_f</td><td>0.007</td><td>0.011</td></tr><tr><td>Horiz.</td><td>NA</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr><tr><td></td><td></td><td></td><td></td><td>A_s</td><td>0.157</td><td>0.225</td></tr><tr><td rowspan="4">18</td><td>Leg</td><td>1.01</td><td>0.060</td><td>0.076</td><td rowspan="4">18</td><td>A_c</td><td>0.177</td><td>0.246</td></tr><tr><td>Diag.</td><td>2.07</td><td>0.027</td><td>0.045</td><td>A_f</td><td>0.009</td><td>0.012</td></tr><tr><td>Horiz.</td><td>NA</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr><tr><td></td><td></td><td></td><td></td><td>A_s</td><td>0.186</td><td>0.258</td></tr><tr><td rowspan="4">24</td><td>Leg</td><td>2.01</td><td>0.076</td><td>0.092</td><td rowspan="4">24</td><td>A_c</td><td>0.605</td><td>0.796</td></tr><tr><td>Diag.</td><td>3.53</td><td>0.042</td><td>0.060</td><td>A_f</td><td>0.030</td><td>0.040</td></tr><tr><td>Horiz.</td><td>NA</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr><tr><td></td><td></td><td></td><td></td><td>A_s</td><td>0.635</td><td>0.836</td></tr></table> Table A.4.2 — Resistance of typical tower panels <table><tr><th>Panel no.</th><th>ϕ</th><th>C_{Nf}</th><th>C_{Nc}</th><th>C_N</th><th>R_w</th></tr><tr><td></td><td>Ice free</td><td></td><td></td><td></td><td>m²</td></tr><tr><td>3</td><td>0.12</td><td>2.80</td><td>1.60</td><td>1.66</td><td>0.20</td></tr><tr><td>9</td><td>0.16</td><td>2.70</td><td>1.55</td><td>1.60</td><td>0.25</td></tr><tr><td>18</td><td>0.11</td><td>2.85</td><td>1.65</td><td>1.71</td><td>0.32</td></tr><tr><td>24</td><td>0.10</td><td>2.90</td><td>1.65</td><td>1.71</td><td>1.09</td></tr><tr><td></td><td>Iced</td><td></td><td></td><td></td><td></td></tr><tr><td>3</td><td>0.19</td><td>2.60</td><td>1.50</td><td>1.55</td><td>0.29</td></tr><tr><td>9</td><td>0.23</td><td>2.45</td><td>1.45</td><td>1.50</td><td>0.34</td></tr><tr><td>18</td><td>0.15</td><td>2.75</td><td>1.60</td><td>1.66</td><td>0.43</td></tr><tr><td>24</td><td>0.16</td><td>2.70</td><td>1.60</td><td>1.66</td><td>1.38</td></tr></table> Full tabulation of these produces total tower resistances as follows. Ice free: $\Sigma R_w = 13 \text{ m}^2$; Ice free: $\Sigma R_w = 17 \text{ m}^2$.	Panel no.	Member	Length	Width		Panel no.	Projected area			Uniced	Iced	Type	Uniced	Iced	3	Leg	1.00	0.042	0.060	3	A_c	0.114	0.178	Diag.	1.41	0.021	0.041	A_f	0.006	0.009	Horiz.	NA	—	—	—	—	—					A_s	0.120	0.187	9	Leg	1.00	0.060	0.078	9	A_c	0.150	0.214	Diag.	1.41	0.021	0.041	A_f	0.007	0.011	Horiz.	NA	—	—	—	—	—					A_s	0.157	0.225	18	Leg	1.01	0.060	0.076	18	A_c	0.177	0.246	Diag.	2.07	0.027	0.045	A_f	0.009	0.012	Horiz.	NA	—	—	—	—	—					A_s	0.186	0.258	24	Leg	2.01	0.076	0.092	24	A_c	0.605	0.796	Diag.	3.53	0.042	0.060	A_f	0.030	0.040	Horiz.	NA	—	—	—	—	—					A_s	0.635	0.836	Panel no.	ϕ	C_{Nf}	C_{Nc}	C_N	R_w		Ice free				m ²	3	0.12	2.80	1.60	1.66	0.20	9	0.16	2.70	1.55	1.60	0.25	18	0.11	2.85	1.65	1.71	0.32	24	0.10	2.90	1.65	1.71	1.09		Iced					3	0.19	2.60	1.50	1.55	0.29	9	0.23	2.45	1.45	1.50	0.34	18	0.15	2.75	1.60	1.66	0.43	24	0.16	2.70	1.60	1.66	1.38
Panel no.	Member				Length	Width		Panel no.	Projected area																																																																																																																																																																																																
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Reference in Part 1	Worked example 1
5.1.1	A.5 Structural response to wind A.5.1 General a) <i>Check for use of the static method.</i> The equivalent static method may be used if: $\frac{7 m_T}{\rho_s R_{WT} \sqrt{(d_B \tau_o)} \left(\frac{5}{6} - \frac{h_T}{H} \right)^2} < 1$ Total tower resistance, $\Sigma R_w = 13 \text{ m}^2$. $R_{WT} \leq \frac{\Sigma R_w}{3} = 4.3 \text{ m}^2$, which from a full version of Table A.4.2 corresponds with panels 0 to 12 inclusive for which $R_{WT} = 4.23 \text{ m}^2$, $m_T = 400 \text{ kg}$ and $h_T = 13.5 \text{ m}$, but not greater than $H/3$. $H/3 = 11 \text{ m}$, hence $h_T = 11 \text{ m}$; $\rho_s = 7\,850 \text{ kg/m}^3$; $\tau_o = 0.001 \text{ m}$; $d_B = 0.75 \times 3 = 2.25 \text{ m}$. Thus $\frac{7 \times 400}{7850 \times 4.23 \times \sqrt{(0.001 \times 2.25)} \left(\frac{5}{6} - \frac{11}{33} \right)^2} = 0.44 < 1$ Therefore the equivalent static method may be used. b) <i>Procedure to be used.</i> The tower panels comply with the constraints of 4.1.3 of Part 1, thus maximum member forces can be derived in accordance with 5.2 of Part 1. Since there are no large ancillaries on the tower (other than those treated as structural members), the only analyses required are under total mean hourly wind load. The total load effects are obtained after the analyses are completed by factoring the forces in each member according to the appropriate gust factors, G, derived according to the height and member type [see d)]. Two wind load cases are considered: 1) with wind normal to a face to produce maximum leg compression (or tension) in the opposite leg; 2) with wind parallel to a face to produce maximum bracing forces. Dead load and ice load have to be dealt with separately, so that due allowance can be made for reversal of derived forces for maximum tension or compression, and for incorporation of alternative values of γ_{DL} in the determination of dead load effects, and of ρ_{iw} in the determination of ice load effects, as well as to ensure that the gust factor, G, is applied only to forces due to wind effects. NOTE If the analyses are carried out by a computer program with automatic self weight procedure, care will be required in the combinations produced to ensure that the appropriate factored load effects are obtained. c) <i>Mean wind load.</i> Values of the maximum mean wind load, $\bar{P}_{TW} = (\rho_a/2) \bar{V}_z^2 \Sigma R_w$ are given in Table A.5.1 for typical panels taking $\rho_a = 1.22 \text{ kg/m}^3$.
5.1.2	
5.2.1	

Reference in Part 1	Worked example 1																																																												
(C.3.5.1 of this part of BS 8100) 5.2.2.2 Figures 5.1 and 5.2 5.2.2.3 Figure 5.3	Table A.5.1 — Mean wind loads on typical panels <table><tr><th>Panel no.</th><th>Height</th><th>$\bar{V}_z$</th><th>R_w</th><th>$\bar{P}_{TW}$</th></tr><tr><td></td><td>m</td><td>m/s</td><td>m²</td><td>kN</td></tr><tr><td colspan="5">Ice free</td></tr><tr><td>3</td><td>27.5</td><td>40.9</td><td>0.20</td><td>0.205</td></tr><tr><td>9</td><td>21.5</td><td>39.5</td><td>0.25</td><td>0.238</td></tr><tr><td>18</td><td>12.5</td><td>36.6</td><td>0.32</td><td>0.261</td></tr><tr><td>24</td><td>1.0</td><td>19.5</td><td>1.09</td><td>0.253</td></tr><tr><td colspan="5">Iced</td></tr><tr><td>3</td><td>27.5</td><td>27.8</td><td>0.29</td><td>0.137</td></tr><tr><td>9</td><td>21.5</td><td>26.9</td><td>0.34</td><td>0.150</td></tr><tr><td>18</td><td>12.5</td><td>24.9</td><td>0.43</td><td>0.163</td></tr><tr><td>24</td><td>1.0</td><td>13.3</td><td>1.38</td><td>0.149</td></tr></table> NOTE 1 Inspection of Table A.5.1 shows that the wind under ice-free conditions governs throughout. This agrees with the general guidance given in C.3.5.1, as follows: $\frac{r_{rw}}{D} = \frac{10}{21} \approx 48\%$ at the top of the tower for the bracings or $\frac{9}{42} \approx 21\%$ at the top of the tower for the legs. Hence, since in all cases $r_{rw}/D < 60\%$, wind with ice is not likely to govern. The loads $\bar{P}_{TW}$ given in Table A.5.1 are used for both load cases: 1) face on to wind; 2) face parallel to wind; since K_θ is identical (= 1.0) for both these conditions for circular-section members (see Figure 4.2 of Part 1). NOTE 2 For flat-sided members or square towers, the loadings would need to be factored to allow for differing K_θ values. d) <i>Gust response factors.</i> Gust response factors are determined as follows. 1) The basic gust response factor is calculated using $G_B = Bj$ as given in 5.2.2.1 of Part 1, using Figures 5.1 and 5.2 of Part 1; typical results are tabulated in Table A.5.2. The value of $(H - z)$ is limited in the upper part of the tower to the minimum value of 10 m when obtaining B, and the consequent value of $(H - z)/H = 10/33 = 0.30$ when obtaining j; these values of $(H - z)$ are given in parenthesis in Table A.5.2. 2) Alternatively, the simplified method, as given in 5.2.2.3 of Part 1, could be used giving G_B from Figure 5.3 of Part 1 as 0.98 for $H = 33$ m which is a slight upper bound on the values given in 5.2.2.2 of Part 1, i.e. the general method.	Panel no.	Height	$\bar{V}_z$	R_w	$\bar{P}_{TW}$		m	m/s	m ²	kN	Ice free					3	27.5	40.9	0.20	0.205	9	21.5	39.5	0.25	0.238	18	12.5	36.6	0.32	0.261	24	1.0	19.5	1.09	0.253	Iced					3	27.5	27.8	0.29	0.137	9	21.5	26.9	0.34	0.150	18	12.5	24.9	0.43	0.163	24	1.0	13.3	1.38	0.149
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(C.3.5.1 of this part of BS 8100)

5.2.2.2
Figures 5.1 and 5.2

5.2.2.3
Figure 5.3

Reference in Part 1	Worked example 1																																																																		
5.2.4 Figure 5.4 (C.5.2.4 of this BS 8100)	3) For bracings, the factor K_q is taken from Figure 5.4 of Part 1, according to $1/ f_q$ where f_q is the proportion of the mean hourly wind shear carried by the bracings at the level considered. For the upper part of the tower with parallel legs, $f_q = 1.0$ since no shear is transmitted through the legs. For the lower part of the tower with sloping legs, f_q is obtained from the analyses under mean wind loading, using: $f_q = \frac{q_b}{\Sigma q}$ where q_b is shear carried by the bracings; Σq is the total shear. Both q_b and Σq are taken appropriate to the level considered and the load direction in each analysis.																																																																		
5.2.3	4) The resulting leg and bracing gust factors are tabulated in Table A.5.2, where $G_1 \text{ is given by } G_B \left\{ 1 + 0.2 \left(\frac{z}{H} \right)^2 \right\}$																																																																		
5.2.4	$G_b \text{ is given by } K_q G_B \left\{ 1 + 0.2 \left(\frac{z}{H} \right)^2 \right\} = K_q G_1$																																																																		
Table A.5.2 — Gust response factors																																																																			
<table><tr><th>Panel</th><th>z</th><th>$H - z$</th><th>$\frac{z}{H}$</th><th>B</th><th>j</th><th>G_B</th><th>f_q</th><th>k_q</th><th>G_1</th><th>G_b</th></tr><tr><td>1</td><td>29.5</td><td>(10)</td><td>0.89</td><td>1.20</td><td>0.80</td><td>0.96</td><td>1</td><td>1</td><td>1.11</td><td>1.11</td></tr><tr><td>3</td><td>27.5</td><td>(10)</td><td>0.83</td><td>1.20</td><td>0.80</td><td>0.96</td><td>1</td><td>1</td><td>1.09</td><td>1.09</td></tr><tr><td>9</td><td>21.5</td><td>11.5</td><td>0.65</td><td>1.16</td><td>0.83</td><td>0.96</td><td>1</td><td>1</td><td>1.04</td><td>1.04</td></tr><tr><td>18</td><td>12.5</td><td>20.5</td><td>0.38</td><td>1.06</td><td>0.92</td><td>0.97</td><td>0.348</td><td>1.14</td><td>1.00</td><td>1.14</td></tr><tr><td>24</td><td>1.0</td><td>32.0</td><td>0.03</td><td>0.98</td><td>1.00</td><td>0.98</td><td>0.337</td><td>1.18</td><td>0.98</td><td>1.15</td></tr></table>		Panel	z	$H - z$	$\frac{z}{H}$	B	j	G_B	f_q	k_q	G_1	G_b	1	29.5	(10)	0.89	1.20	0.80	0.96	1	1	1.11	1.11	3	27.5	(10)	0.83	1.20	0.80	0.96	1	1	1.09	1.09	9	21.5	11.5	0.65	1.16	0.83	0.96	1	1	1.04	1.04	18	12.5	20.5	0.38	1.06	0.92	0.97	0.348	1.14	1.00	1.14	24	1.0	32.0	0.03	0.98	1.00	0.98	0.337	1.18	0.98	1.15
Panel	z	$H - z$	$\frac{z}{H}$	B	j	G_B	f_q	k_q	G_1	G_b																																																									
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2.5	A.6 Strength assessment A.6.1 General a) <i>Required reliability.</i> To achieve the required reliability with respect to strength: $\Sigma F_{\max} \leq Q_d$ where $\Sigma F_{\max}$ is the maximum total force in the member; Q_d is the design strength of the member given by: Q_k / γ_m where Q_k is the characteristic strength of the member. b) <i>Design strengths.</i> The values of Q_k, and hence Q_d, may be obtained from appropriate characteristic strengths, or may be conservatively obtained from DD 133, according to type and slenderness, as follows. NOTE The detailed calculations on the interpretation of DD 133 are not given here as the document is a Draft for Development.
2.5	

Reference in Part 1	Worked example 1																																												
	<table><tr><th>Member</th><th>Member type</th><th>Q_k</th><th>γ_m</th><th>Q_d</th></tr><tr><td rowspan="3">Bracings</td><td>mm</td><td>kN</td><td></td><td>kN</td></tr><tr><td>21.3×3.2</td><td>8.8</td><td>1.1</td><td>8.0</td></tr><tr><td>26.9×3.2</td><td>9.8</td><td>1.1</td><td>8.9</td></tr><tr><td></td><td>42.4×3.2</td><td>15.3</td><td>1.1</td><td>13.9</td></tr><tr><td rowspan="4">Legs</td><td>42.4×4.0</td><td>35</td><td>1.1</td><td>32</td></tr><tr><td>60.3×4.0</td><td>95</td><td>1.1</td><td>86</td></tr><tr><td>60.3×5.0</td><td>138</td><td>1.1</td><td>125</td></tr><tr><td>76.1×5.0</td><td>214</td><td>1.1</td><td>195</td></tr></table>					Member	Member type	Q_k	γ_m	Q_d	Bracings	mm	kN		kN	21.3×3.2	8.8	1.1	8.0	26.9×3.2	9.8	1.1	8.9		42.4×3.2	15.3	1.1	13.9	Legs	42.4×4.0	35	1.1	32	60.3×4.0	95	1.1	86	60.3×5.0	138	1.1	125	76.1×5.0	214	1.1	195
	Member	Member type	Q_k	γ_m	Q_d																																								
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		76.1×5.0	214	1.1	195																																								
c) <i>Strength check.</i> The safety assessment in the form of a strength check is most conveniently set out in the form of stress ratios, <i>SR</i> , where:																																													
$SR = \frac{\Sigma F_{\max}}{Q_d}$																																													
<i>The required reliability is then achieved provided $SR \leq 1.0$.</i> The levels of <i>SR</i> also indicate the capacity of the tower for carrying extra loadings, either in the form of ancillary attachments, aerials, dishes, etc., or alternative sitings, such as on hills or in more severe meteorological zones. The <i>SR</i> values are as follows.																																													
<table><tr><th>Panels</th><th>Member (CHS)</th><th>$\Sigma F_{\max}$</th><th>Q_d</th><th>Maximum stress ratio (<i>SR</i>)</th></tr><tr><td rowspan="4"><i>Bracings</i> 1 to 9 10 to 18 10 to 24</td><td>mm</td><td>kN</td><td>kN</td><td></td></tr><tr><td>21.3×3.2</td><td>5.5</td><td>8.0</td><td>0.69</td></tr><tr><td>26.9×3.2</td><td>4.6</td><td>8.9</td><td>0.52</td></tr><tr><td>42.4×3.2</td><td>6.7</td><td>13.9</td><td>0.48</td></tr><tr><td rowspan="4"><i>Legs</i> 1 to 3 4 to 9 10 to 18 19 to 24</td><td>42.4×4.0</td><td>20.4</td><td>32</td><td>0.64</td></tr><tr><td>60.3×4.0</td><td>56.9</td><td>86</td><td>0.66</td></tr><tr><td>60.3×5.0</td><td>85.6</td><td>125</td><td>0.68</td></tr><tr><td>76.1×5.0</td><td>137.8</td><td>195</td><td>0.71</td></tr></table>					Panels	Member (CHS)	$\Sigma F_{\max}$	Q_d	Maximum stress ratio (<i>SR</i>)	<i>Bracings</i> 1 to 9 10 to 18 10 to 24	mm	kN	kN		21.3×3.2	5.5	8.0	0.69	26.9×3.2	4.6	8.9	0.52	42.4×3.2	6.7	13.9	0.48	<i>Legs</i> 1 to 3 4 to 9 10 to 18 19 to 24	42.4×4.0	20.4	32	0.64	60.3×4.0	56.9	86	0.66	60.3×5.0	85.6	125	0.68	76.1×5.0	137.8	195	0.71		
Panels	Member (CHS)	$\Sigma F_{\max}$	Q_d	Maximum stress ratio (<i>SR</i>)																																									
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	76.1×5.0	137.8	195	0.71																																									
Hence the tower is satisfactory for the intended use at the site considered. There is a lower bound limit of about 40 % increase in load before the size of the members would need to be increased.																																													
3.4.2.1 5.1.5 5.5	A.6.2 Vortex excitation a) <i>Overall response.</i> The cylindrical fibreglass shroud may cause vortex-excited vibrations and this needs to be checked in accordance with 5.5 of Part 1.																																												

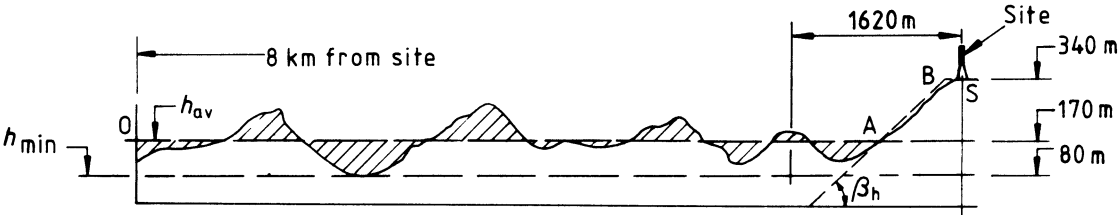
Reference in Part 1	Worked example 1
5.5.1 5.5.2 Figure 5.8	b) <i>Critical wind speed in fundamental mode.</i> The lowest critical wind speed occurs at: $V_{cr1} = \frac{n_1 D}{S}$ For $n_1 = 1.9$ Hz (from an initial estimate of frequency); $S = 0.2$ for circular cylinders; $V_{cr1} = 6.6$ m/s. This is less than $1.3 \bar{V}_z (= 55 \text{ m/s})$, thus the tower is subject to vibrations induced through vortex shedding. The amplitudes and hence member forces so induced can be estimated by considering the tower under the influence of aerodynamic loads given by: $0.9 \rho_a \bar{V}_e^2 k_e C' D_z \sin 2 \pi n t \text{ per unit height}$ In order to obtain the largest force, the value $\bar{V}_e^2 k_e$ has to be maximized. This can be done using Figure 5.8 of Part 1 for various values of k_e. The maximum value occurs at $k_e = 0.75$, giving: $\frac{1.3 \bar{V}_e}{V_{cr1}} = 1.2$ $\bar{V}_e = \frac{1.2}{1.3} \times 6.6$ $= 6.1 \text{ m/s}$ Hence the maximum value that the fluctuating aerodynamic load can have in the fundamental mode is: $0.9 \times 1.22 \times 6.1^2 \times 0.75 \times 0.3 \times 0.7 \times 1.0 = 6.5 \text{ N/m}$ This is equivalent to a total load on the cylinder of 0.019 kN. The aerodynamic load effects may be assessed initially using the periodic dynamic magnifier $\pi/\Sigma\delta$. For crosswind response, aerodynamic damping, $\delta_a = 0$. Using Table E.1 and Table E.2 of Part 1, δ_T is taken as 0.06 and $K_\delta = 2$, for spread footing on medium soil. Hence $\Sigma\delta = K_\delta \delta_T = 0.12$ Hence the effective dynamic load $\simeq \frac{\pi}{0.12} \times 0.019 = 0.5 \text{ kN}$. The total equivalent static load on the cylinder is given by: $(1 + G) \bar{P}_{TW} = (1 + 1.15) \times 1.56 = 3.35 \text{ kN}$ Hence vortex shedding forces are approximately 15 % of this and may be neglected.

Reference in Part 1	Worked example 1																										
3.4.2.1 3.3 3.4.2.2	c) <i>Further modes.</i> Due to the low $\bar{V}_{cr}$ in the fundamental mode, the tower has to be checked for higher modes. The next two modes of the tower have frequencies estimated as: $n_2 = 6.9 \text{ Hz}$ $n_3 = 20.4 \text{ Hz}$ Thus $V_{cr2} = \frac{6.9 \times 0.7}{0.2} = 24 \text{ m/s} < 1.3 \bar{V}_z (= 55 \text{ m/s})$ and $\bar{V}_{e\max} = \frac{1.2}{1.3} \times 24 = 22 \text{ m/s for } K_e = 0.75$ The maximum value of fluctuating aerodynamic force in the second mode is $0.9 \times 1.22 \times 22.0^2 \times 0.75 \times 0.3 \times 0.7 \times 1.0 = 84 \text{ N/m}$, which is equivalent to a total effective dynamic load of $\approx 3 \times 0.084 \times \pi/0.12 = 6.6 \text{ kN}$, which is approximately twice the total equivalent static load on the cylinder. However, this force (and to a lesser extent that from the fundamental frequency) will be reduced by mode shape considerations, when comparing with the equivalent static loads applied to the tower. Consequently, the effects of vortex shedding may be a governing criterion for the design of the strength of the tower, and a simple dynamic analysis should be made to derive load effects [see f)]. For the third frequency of vibration: $V_{cr3} = 71 \text{ m/s} > 1.3 \bar{V}_z (= 55 \text{ m/s})$ Thus vortex excitation should not occur in this or any higher mode, but this should also be checked by dynamic analysis in this case [see f)]. d) <i>Fatigue.</i> The effect of the first and second modes of vortex excitation may need consideration for fatigue, in accordance with 3.4.2 of Part 1, and for serviceability, in accordance with 3.3 of Part 1 (see A.6.3). e) <i>Individual member response.</i> Slender tubular members may be subject to vortex shedding at low wind speeds. The slenderness of the members in this tower are such that the resulting stresses will be relatively small, but may need to be considered for fatigue. f) <i>Results from simple dynamic analysis.</i> The results from simple dynamic analysis are as follows. <table><tr><th rowspan="2">Mode</th><th rowspan="2">Frequency</th><th rowspan="2">V_{cr}</th><th rowspan="2">δ</th><th colspan="2">Ratio dynamic/static</th></tr><tr><th>Shear</th><th>Moment</th></tr><tr><td>Mode 1</td><td>1.7 Hz</td><td>6.1 m/s</td><td>13 mm</td><td>0.08</td><td>0.09</td></tr><tr><td>Mode 2</td><td>7.4</td><td>26.0</td><td>5</td><td>0.35</td><td>0.40</td></tr><tr><td>Mode 3</td><td>18.9</td><td colspan="4">66.1 $\rightarrow$ exceeds $1.3 \bar{V}_z$</td></tr></table> The maximum load effects are less than 40 % of the static design criteria, and hence will not govern.	Mode	Frequency	V_{cr}	δ	Ratio dynamic/static		Shear	Moment	Mode 1	1.7 Hz	6.1 m/s	13 mm	0.08	0.09	Mode 2	7.4	26.0	5	0.35	0.40	Mode 3	18.9	66.1 $\rightarrow$ exceeds $1.3 \bar{V}_z$			
	Mode					Frequency	V_{cr}	δ	Ratio dynamic/static																		
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	Mode 3	18.9	66.1 $\rightarrow$ exceeds $1.3 \bar{V}_z$																								

Reference in Part 1	Worked example 1
2.6	A.6.3 Serviceability
3.3	a) <i>General</i> . The serviceability criteria have been specified as a limiting deflection of 50 mm, not to be exceeded for more than 20 h per year, i.e. $\Delta_s = 50$ mm, the specified deflection limit.
5.1.4	b) <i>Downwind deflections</i> . From the analysis under mean hourly loading, the maximum deflection, $\delta = 78$ mm at the level of the shroud.
5.2.5	From 5.2.5 of Part 1 $\Delta = (1 + G_{B \text{ base}}) \times \delta = 1.98 \times 78 = 154$ mm Hence
3.3.2	$\frac{\bar{V}_s}{\bar{V}_k} = \gamma_v \sqrt{\left(\frac{\Delta_s}{\Delta}\right)} = 1.15 \sqrt{\left(\frac{50}{154}\right)} = 0.66$
Figure 3.6	This limiting deflection is independent of the angle of wind incidence, hence from Figure 3.6 of Part 1, the number of hours per year that the aerial will be out of service is 18 h and hence the criteria are satisfied, and the wind speed $\bar{V}_s$ at which the limit occurs = $0.66 \times 30.9 = 20.4$ m/s.
3.1.5	a) <i>Crosswind deflections (vortex shedding)</i> . The forces due to vortex excitation have been derived from a simple dynamic analysis, the results giving the crosswind deflection, as
3.3.3	
5.5.2	$\delta_1 = 13$ mm in the first mode $\delta_2 = 5$ mm in the second mode [see A.6.4 b)] i.e. $\Delta'_{\max} = 13$ mm and hence $\Delta_s (= 50$ mm) is never exceeded. Therefore it is satisfactory.
	NOTE If an aerial, more sensitive to deflection, was proposed such that $\Delta_s = 12$ mm, but the period of unserviceability were increased to 600 h per year, the following results would be obtained.
3.3.2	1) Downwind: $\frac{\bar{V}_s}{\bar{V}_k} = 1.15 \sqrt{\left(\frac{12}{154}\right)} = 0.32$
Figure 3.6	The period out of service (see Figure 3.6 of Part 1) = 2 100 h per year.
	2) Crosswind: $\bar{V}_s = 6.1$ m/s in the first mode, for which $\Delta' = 13$ mm is greater than Δ_s (12 mm), giving: $\frac{\bar{V}_s}{\bar{V}_k} = \frac{6.1}{30.9} = 0.20$
Figure 3.8 b)	From Figure 3.8 of Part 1, at 240° annual occurrence = 290 h, and total time out of service = $290 \times 6.5 = 1\ 880$ h/year
3.3.3	For higher modes, $\delta < \Delta_s$ and need not be considered. 3) Total period out of service = 3 980 h = 5 months which is unsatisfactory.

Reference in Part 1	Worked example 1
3.4.1 3.4.2 3.4.2.1 3.3.3 3.4.2.2	A.6.4 Fatigue a) <i>In-line vibrations</i>. Since the tower is of steel grade 43 complying with BS 4360, i.e. yield stress ≤ 355 N/mm², the fatigue life can be assumed to exceed 50 years if bolted connections are used. If welded, the fatigue life may still be assumed to be 30 years, without further checks, and is therefore satisfactory for the intended service life. b) <i>Vortex-excited vibrations</i>. The vortex-excited vibrations are as follows. 1) <i>Overall response</i>. Duration of first mode of vibration: $\frac{\bar{V}_s}{\bar{V}_k} = \frac{6.6}{30.9} = 0.21$ Number of hours per year = $6.5 \times 290 = 1\,890$ h/year But stress levels $< 10\%$ of maximum equivalent static stress. Duration of second mode of vibration: $\frac{\bar{V}_s}{\bar{V}_k} = \frac{22}{30.9} = 0.71$ Number of hours per year = $6.5 \times 13 = 85$ h/year But stress levels $\leq 40\%$ of maximum equivalent static stress. These values can be checked against the appropriate fatigue criteria, but should not result in any reduction on the 30 year life. 2) <i>Individual member response</i>. The stress range will be small, and need not be considered further for this particular tower. A.6.5 Summary A.1 to A.6.4 complete the design process for the intended use of the tower at the particular site. However this tower may also need to be assessed for use on a hill site and for use abroad. The preliminary calculations required are set out in A.7 and the results are summarized as follows.
Figures 3.4 and 3.5 3.2.2	a) <i>Alternative site on hill</i>. The comparison of wind loads shows that, for the particular hill site chosen, the wind loads (and hence member forces) increase by an overall factor greater than two, and hence a full redesign would be required for such a site. The maximum increase in wind speed due to a hill that could be accommodated by the present design is only of the order of $\sqrt{1.4} = 1.2$. Inspection of Figures 3.4 and 3.5 of Part 1 and use of 3.2 of Part 1 show that the factor on wind speed at any height z is given by: $\frac{K_\mu}{\left(\frac{z}{10}\right)^\mu}$ This can be used to check initially the acceptability of any hill site, e.g. for a 30 m high hill of slope $\beta = 6^\circ$, at a height of 20 m: $\frac{K_\mu}{\left(\frac{z}{10}\right)^\mu} \approx \frac{1.27}{2^{0.007}} = 1.21$ This may be shown to be satisfactory with further detailed study. b) <i>Alternative overseas site</i>. This shows an increase in wind load of about 1.6 and the basic tower could only be used with most members increased as required following further analyses.

Reference in Part 1	Worked example 1
	A.7 Alternative sites A.7.1 Alternative site on hill For an alternative site on a hill the following preliminary calculations are required. a) Assume the tower was sited on a nearby hill as shown in the following figure with the site level = 340 m.



Appendix D

$h_{av} = 170\text{ m}$, $h_{min} = 80\text{ m}$
 $18(h_{av} - h_{min}) = 1\,620\text{ m}$
The hill is defined by OABS.
Hence the height of the hill, $H = 340 - 170 = 170\text{ m}$

$$\text{Slope} = \sin^{-1} \left(\frac{170}{1000} \right) = 9.8^\circ; x \approx 80\text{ m}$$

Figure 3.5

Hence:

$$K_\mu = 1.64; K_{\mu x} = 1.64 - \frac{80 \times 0.64}{18 \times 170} = 1.62$$

$$\mu = 0.101; \mu_x = 0.101 \left(1 - \frac{80}{18 \times 170} \right) = 0.098$$

Figure 3.4

so $\bar{V}_r K_{\mu x} = 57.5\text{ m/s}$; $\alpha - \mu_x = 0.042$
and $\bar{V}_z = 57.5 (z/10)^{0.042}$ for $z \geq 10$
 $28.8 (1 + z/10)$ for $z < 10$

b) A comparison can be made with that for the level site. Table A.7.1 gives the ratio of factored wind speeds and also the ratio of wind speeds squared to give an indication of the increase in loading.

Table A.7.1 — Comparison of wind profiles

Height z	$\bar{V}_{zh}$ (hill)	$\bar{V}_{zf}$ (flat)	$\frac{\bar{V}_{zh}}{\bar{V}_{zf}}$	$\left(\frac{\bar{V}_{zh}}{\bar{V}_{zf}} \right)^2$
m	m/s	m/s		
27.5	60.0	40.9	1.47	2.15
21.5	59.4	39.5	1.50	2.26
12.5	58.0	36.6	1.59	2.51
1.0	31.8	19.5	1.63	2.66

c) Inspection of column 5 of Table A.7.1 indicates that for the identical tower situated on the hill, member forces will increase by a factor of approximately **2.3**. Since resistances and gust factors are unchanged, the total member forces will be likewise factored by **2.3**. Inspection of the strength checks given in **A.6.1 c)** indicates that the tower members will be under-strength, and increased sizes are required.

The deflections will also increase, and hence the serviceability criteria will need to be reassessed. Additional analyses should be made, as required.

Reference in Part 1	Worked example 1
3.1.1	A.7.2 Alternative site with limited wind data The tower is a standard design developed for use both in the UK and overseas. An example of an alternative location with limited wind data is as follows. <i>Basic wind speed</i>
3.1.2	a) $\bar{V}_B$ has to be derived from a statistical analysis since there are no appropriate official Meteorological Office wind maps of the site. The site is near a local airport where the mean hourly wind speed data has been recorded for only 13 years, consequently a coefficient of variation of 0.2 has been taken. Thus for the recording station, for a height of 10 m:
A.1.5	$\bar{V}_{av} = 32.0 \text{ m/s}, \nu = 0.20$
(C.3.1.2 of this Part of BS 8100)	$\frac{\bar{V}_{max}}{\bar{V}_{av}} = 1 + 2.6\nu \text{ (50 year return period)}$ Hence: $\bar{V}_{max} = 1.52 \times 32.0$ $= 48.6 \text{ m/s}$
A.1.6	The airport terrain is category II thus $\bar{V}_{max}$ has to be divided by the appropriate value of K_R to obtain $\bar{V}_B$, the basic wind speed: $\bar{V}_B = \bar{V}_{max}/K_R$ $= 48.6/1.10 = 44.2 \text{ m/s}$ No correction for altitude is required since the airport and site are the same height above sea level.
Appendix C	b) Data from similar climatic zones have been examined and a gradient wind speed of 70 m/s has been used as a probable value. The gradient wind height was 1 000 m for the zone data.
C.2	Using Figure C.2 of Part 1, a value for the gradient wind speed reduction factor, K_g , is obtained. $K_g = 0.57 \text{ (for site 10 km from the coast and gradient wind height of 1 000 m)}$ Thus: $\bar{V}_B = 0.57 \times 70$ $= 39.9 \text{ m/s}$
Figure C.2	c) An additional data source is a code for which a design wind speed of 69 m/s is appropriate. This is a 2 s to 3 s gust wind speed, and the terrain category of that code corresponds with the basic terrain category of Part 1. $V_t/\bar{V} = 1.55 \text{ for 2 s to 3 s gust, giving:}$ $\bar{V} = \bar{V}_B = 69/1.55 = 44.5 \text{ m/s}$
(Table C.3.2 of this Part of BS 8100)	d) Thus $\bar{V}_B$ has been adopted as that obtained from site measurements since the value has been approximately confirmed using wind speeds from remote stations and gradient wind data.
Figure A.1	Consequently, the predicted value from a) will be used, and $\bar{V}_B$ is taken as 44.2 m/s.

Reference in Part 1	Worked example 1																														
3.1.3 3.1.4 Table 3.1 Figure 2.1 3.2.5 3.2.1	e) The terrain for the site in category IV and the ground is essentially flat. A wind profile may be developed for the site using the parameters from Part 1. $K_d = 1.0$ (for no icing) Terrain category IV, therefore: $K_R = 0.86$; $h_e = 2$; $\alpha = 0.19$. $\gamma_v = 1.15$ (using the same criteria as the East Riding site) NOTE There may need to be a modification to γ_m (see notes in A.2.3) $V_r = \gamma_v K_d K_H \bar{V}_B$ $= 1.15 \times 1.0 \times 0.86 \times 44.2$ $= 43.7 \text{ m/s}$ The wind profile is: $\bar{V}_z = 43.7 \left(\frac{z - 2}{10} \right)^{0.19} \text{ for } z \geq 12 \text{ m};$ $= 21.85 \left(1 + \frac{z}{12} \right) \text{ for } z < 12 \text{ m}.$ A comparison between the wind speeds at the alternative site and at East Riding is as follows. <table><tr><th>Height</th><th>$\bar{V}_{zT}$ (alternative)</th><th>$\bar{V}_{zR}$ (E.Riding)</th><th>$\frac{\bar{V}_{zT}}{\bar{V}_{zR}}$</th><th>$\left(\frac{\bar{V}_{zT}}{\bar{V}_{zR}} \right)^2$</th></tr><tr><td>m</td><td>m/s</td><td>m/s</td><td></td><td></td></tr><tr><td>27.5</td><td>53</td><td>40.9</td><td>1.29</td><td>1.66</td></tr><tr><td>21.5</td><td>50</td><td>39.5</td><td>1.27</td><td>1.62</td></tr><tr><td>12.5</td><td>41</td><td>36.6</td><td>1.16</td><td>1.35</td></tr><tr><td>1.0</td><td></td><td>19.5</td><td></td><td></td></tr></table> f) The tower has been designed as a standard tower which can be used in several locations. At its East Riding site, it is approximately 70 % fully stressed. Thus at the alternative site, the same tower could only be used by strengthening critical members since the increased wind loading would produce a stress increase of about 60 %, giving a maximum stress ratio $\approx 1.6 \times 0.7 = 1.12$.	Height	$\bar{V}_{zT}$ (alternative)	$\bar{V}_{zR}$ (E.Riding)	$\frac{\bar{V}_{zT}}{\bar{V}_{zR}}$	$\left(\frac{\bar{V}_{zT}}{\bar{V}_{zR}} \right)^2$	m	m/s	m/s			27.5	53	40.9	1.29	1.66	21.5	50	39.5	1.27	1.62	12.5	41	36.6	1.16	1.35	1.0		19.5		
Height	$\bar{V}_{zT}$ (alternative)	$\bar{V}_{zR}$ (E.Riding)	$\frac{\bar{V}_{zT}}{\bar{V}_{zR}}$	$\left(\frac{\bar{V}_{zT}}{\bar{V}_{zR}} \right)^2$																											
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12.5	41	36.6	1.16	1.35																											
1.0		19.5																													

Appendix B Worked example 2

Reference in Part 1	Worked example 2
	B.1 Site and tower details B.1.1 General The tower is an existing structure and is located in the New Forest in a wooded area. The ground is gently undulating and may be treated as level ground. The altitude of the site is 60 m AMSL. The structure is a square tower 135 m high and is to be considered as a manned broadcasting tower. It is located alongside a railway, but otherwise is away from any form of habitation. The tower is to be checked with both existing and proposed dishes and aerials mounted on it. B.1.2 Form of structure The general form of the tower is shown in Figure B.1, which also indicates the panels for which specific calculations are made in relation to this example. All sections used are equal angles of various sizes as shown.
2.1 2.1.2 2.2 2.2.2 2.2.3.2 2.2.3.3 2.3 2.3.2 Appendix F 2.4 2.4.1 Figure 2.1	B.2 Performance requirements B.2.1 Service life and safety factors Long-term planning will lead to a replacement for this tower in about 20 years time. Allowing for some change in planning times, the required service life is taken as 25 years. The safety factors appropriate to the required reliability of the tower are derived as given in B.2.2 to B.2.4. B.2.2 Classification of required reliability a) <i>Environmental conditions</i>. The environmental category is that of “adjacent to railway”. b) <i>Economic consequences or usage</i>. A value of $g = 4.5$ has been determined for the ratio of consequential costs of failure to present costs, taking account of the costs of replacement, removal of the failed tower, loss of revenue during inoperation and contingency costs such as third-party claims. The alternative usage category is “broadcasting”. B.2.3 Classification of quality The tower complies with all the recommendations of 2.3.2 of Part 1 in that it had been designed and fabricated in accordance with the appropriate British Standards and is being independently appraised and inspected. The construction and quality control of materials was in accordance with the relevant British Standards. The tower is inspected every 2 years for damage, corrosion or other defects. As a result the tower is deemed to be class A. NOTE Appendix F of Part 1 may also be used to assess the tower class (see Appendix A for a typical example). B.2.4 Safety factors a) <i>Wind speed and ice thickness factors</i>. From Figure 2.1 of Part 1: 1) from the environmental category of “adjacent to railway”: $\gamma_v = 1.21$

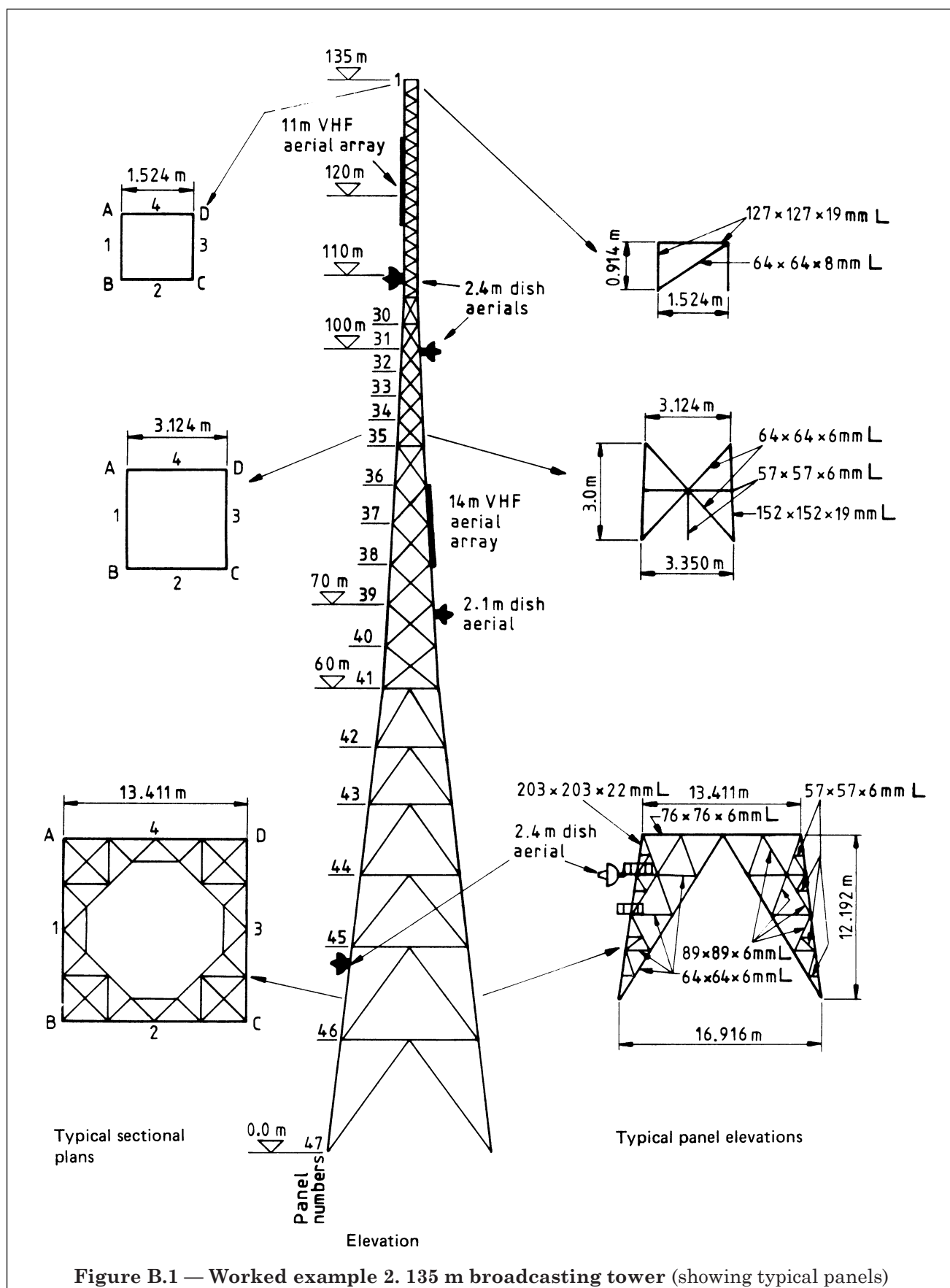
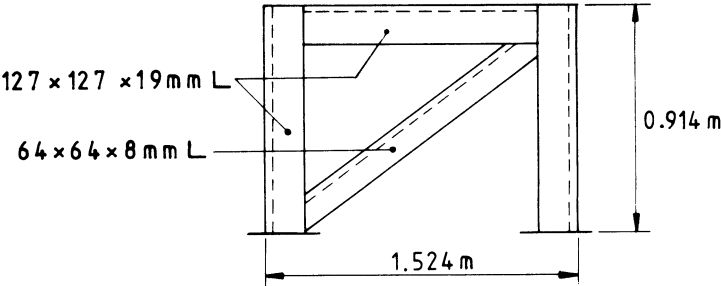
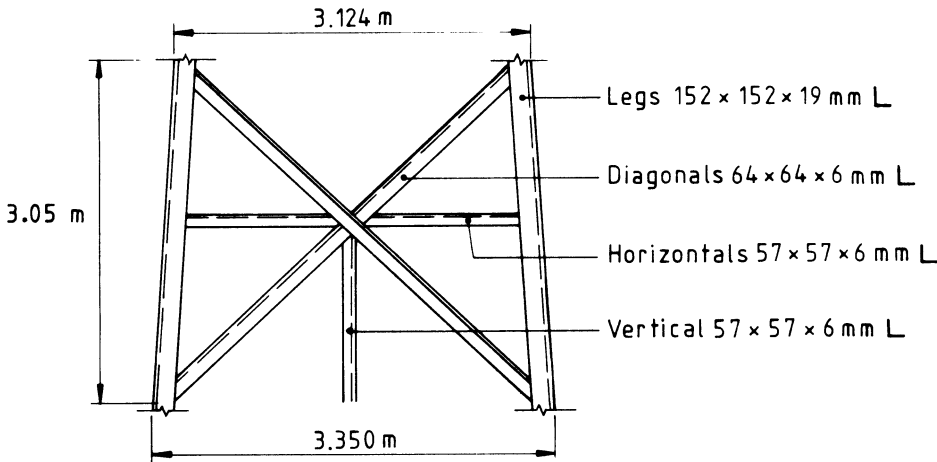


Figure B.1 — Worked example 2. 135 m broadcasting tower (showing typical panels)

Reference in Part 1	Worked example 2
Figure 2.1 (C.2.5 of this Part of BS 8100) 2.4.2 Figure 2.1 (or Table C.2.2 of this Part of BS 8100) 2.5 Figure 2.1	2) from the economic consequences: $\log_{10}(gi_s) = (4.5 \times 25) = 2.05$ which gives $\gamma_v = 1.17$ to 1.22 (This is further to the right than “broadcasting” usage which gives $\gamma_v = 1.15$) In this case (1) and (2) give similar values, and the wind speed factor will be taken as $\gamma_v = 1.21$. b) <i>Dead load factors</i>. Again using the same governing performance requirement from a), the dead load factors from Figure 2.1 of Part 1 are: $\gamma_{DL} = 1.06$ when dead load effects increase wind load effects; 0.90 when dead load effects decrease wind load effects. c) <i>Partial safety factor on strength</i>. From Figure 2.1 of Part 1: $\gamma_m = 1.10$ for quality class A.
3.1 3.1.2 Figure 3.1 3.1.3 3.1.4 Note 3 to Table 3.1 Note 4 to Table 3.1 (C.3.7 of this Part of BS 8100) (Figure C.3.33 of this Part of BS 8100) (Figure C.3.31 of this Part of BS 8100) (Figure C.3.32 of this Part of BS 8100) Table 3.1	B.3 Meteorological parameters B.3.1 General a) <i>Basic wind speed</i>. Using Figure 3.1 of Part 1 wind speed is given as 21.4 m/s at 10 m AMSL. The site is 60 m AMSL and hence the basic mean hourly wind speed is: $\bar{V}_B = 21.4 \times 1.06 = 22.7$ m/s b) <i>Wind direction factor</i>. The site is such that there is no marked difference in terrain with direction near the site nor are there topographic effects requiring factoring of wind speed in any particular direction. Also, the structural configuration of the tower is symmetrical, and the overall wind resistance will be sensibly constant for each face or diagonal wind direction. Hence, for wind without ice, K_d will be taken as 1.0, and for wind with ice K_d will be taken as 0.85. c) <i>Terrain roughness parameters</i>. The wooded terrain surrounding the site extends generally 10 km in the westerly, northerly and easterly directions, but the coast is situated 3 km away to the south at one point. Also there is an area of cleared farmland extending approximately 2 km, starting 2 km away to the north-west, and hence the effects of terrain roughness should be considered, since the site has rougher terrain within a few kilometres upwind, than that outside this area for upwind directions to the north-west and to the south. Using C.3.7 which gives guidance on terrain changes, the effect of the open farmland to the north-west can be neglected since the extent of the open area is less than the distance to the site with category V terrain both inside and outside, i.e. complying with situation b) of Figure C.3.33 of this Part of BS 8100. For the coastal effect to the south, use can be made of Figure C.3.31 and Figure C.3.32 of this Part of BS 8100. For a 3 km distance to the change of terrain, the mean hourly ABL is well above the top of the tower, and the gust ABL intersects the tower at a level of 120 m (see Figure C.3.32 of this Part of BS 8100). Hence the tower is virtually entirely encompassed by the wind profile of terrain category V and this is used for design. From Table 3.1 of Part 1 the terrain parameters are: $K_R = 0.72$; $\alpha = 0.23$ $h_e = 10$ m

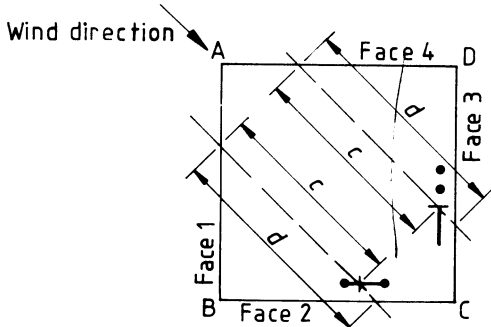
Reference in Part 1	Worked example 2
3.2.1 Figure 3.3 Table 3.1 (Figure C.3.1 of this Part of BS 8100) 3.1.5	Part 1 permits interpolation for values of h_o and K_R through the use of Figure 3.3 of Part 1. The terrain description given in Table 3.1 of Part 1 for category V describes the site conditions suitably, so Figure 3.3 of Part 1 has not been used; the parameters and terrain description are confirmed from Figure C.3.1 of this Part of BS 8100. d) <i>Site reference wind speed</i>. The mean hourly wind speed at the site at a height of 20 m, i.e. 10 m above the effective level of surface obstructions, is: $\bar{V}_r = \gamma_v K_d K_R \bar{V}_B$ $= 1.21 \times 1.0 \times 0.72 \times 22.7 = \underline{19.8} \text{ m/s}$ For serviceability, if required: $\bar{V}_r = K_R \bar{V}_B$ $= 0.72 \times 22.7 = \underline{16.3} \text{ m/s}$
3.2 3.2.1	B.3.2 Wind profile The site is on level ground, therefore 3.2.1 of Part 1 is used to obtain the wind profile. The mean wind speed $\bar{V}_z$ at any required height z above the site ground level, i.e. the base of the tower, is determined for ice-free conditions from: $\bar{V}_z = 19.8 \left(\frac{z}{10} - 1 \right)^{0.23} \quad \text{for } z \geq 20 \text{ m;}$ $= 9.9 \left(1 + \frac{z}{20} \right) \quad \text{for } z < 20 \text{ m.}$
3.3 5.5	B.3.3 Serviceability Serviceability is dealt with in 3.3 of Part 1 and is considered independently from the derivation of loads and strength (safety) assessment. This calculation is carried out after the analyses are made together with any assessment of vortex shedding (see 3.3.3 and 5.5 of Part 1). These checks have not been carried out for this tower, but would be similar to those detailed in Appendix A, A.6.3.
3.4 5.5	B.3.4 Fatigue Fatigue is dealt with in 3.4 of Part 1 and is considered independently from the derivation of loads and strength (safety) assessment. This calculation is carried out after the analyses are made together with any assessment of vortex shedding (see 3.4.2 and 5.5 of Part 1). These checks have not been carried out for this tower, but would be similar to those detailed in Appendix A, A.6.4.
3.5 3.5.2 Figure 3.9	B.3.5 Ice loading a) <i>Basic ice thickness</i>. The ice thickness appropriate to the site are taken from the map in Figure 3.9 of Part 1. At the site of the tower in the New Forest, the following thicknesses are obtained: $r_o = 50 \text{ mm (in the absence of wind);}$ $r_w = 0 \text{ (with wind).}$ The altitude of the tower top is $a_T = a_o + H$, giving $a_T = 60 + 135 = 195 \text{ m}$, which is less than 200 m and hence there is no increase on r_o and r_w. Also $k_i = 1.0$ since all sections are angles, and $r_B = r_o$ or r_w, as appropriate.
3.5.3 3.5.4 3.5.1	b) <i>Reference ice thickness, ice weight and combinations</i>. For the case of wind with ice, $r_r = 0$, and hence no further considerations of wind with ice are required, and the only combination to be considered is wind only (ice free).

Reference in Part 1	Worked example 2
4.1 4.1.1 4.1.2 4.2 4.2.1 Figure 4.1 Figure 4.2 4.2.2 Figure 4.3 a) 4.2.1 a) 4.2.1 b)	B.4 Wind resistance B.4.1 General The tower has already been divided into 47 panels in the reference data provided, such that each panel encompasses a set of primary diagonal cross-bracing. NOTE More typically, however, some of the upper panels would be aggregated up, to about 6 m heights of tower, since $0.05 \times 135 = 6.75$ m, halving the total number of panels for the purposes of wind resistance calculation from 47 to about 23. Three panels have been selected to demonstrate the use of Part 1. These three panels (numbers 1, 35 and 46 as indicated in Figure B.1) show the effects of the various ancillaries which are likely to be encountered in the full analysis of the tower. The panels have been selected so as to demonstrate the three different methods of calculating panel resistances and tower response. B.4.2 Panel 1 There are no ancillaries on panel 1, thus it complies with 4.1.2 of Part 1. Consequently the total resistance can be derived in accordance with 4.2 of Part 1.  The panel is composed of flat sections only, so: $A_s = A_f = (0.127 \times 0.914 \times 2) + \{(1.524 - 0.254) \times 0.127\} + (1.494 \times 0.064) = 0.49 \text{ m}^2$ $\phi = \frac{A_s}{ht} = \frac{0.49}{0.914 \times 1.524} = 0.35$ The wind incidence factor, K_θ, is derived from Figure 4.2 of Part 1 for $A_f/A_F = 1.0$ and $\phi = 0.35$. For wind on face ($\theta = 0^\circ$), $K_\theta = 1.0$. For wind on diagonal ($\theta = 45^\circ$), $K_\theta = 1.19$. <i>Drag coefficients</i> From $C_N = C_{Nf} \frac{A_f}{A_F} + C_{Nc} \frac{A_c}{A_F} + C_{Nc'} \frac{A_{c'}}{A_F}$ $A_c = A_{c'} = 0$, and $A_f = A_F$ Hence $C_N = C_{Nf} = 2.36$ $R_W = R_T = K_\theta C_N A_s$ For $\theta = 0^\circ$ $R_W = 1.0 \times 2.36 \times 0.49 = 1.16 \text{ m}^2$ and $R_x = R_W = 1.16 \text{ m}^2$ For $\theta = 45^\circ$ $R_W = 1.19 \times 2.36 \times 0.49 = 1.39 \text{ m}^2$ and $R_x = R_W = 1.39 \text{ m}^2$

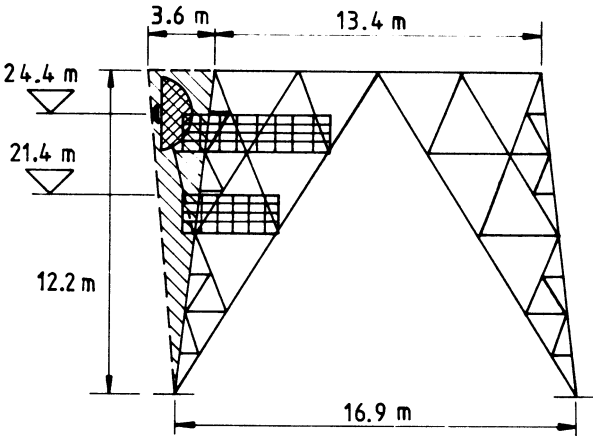
Reference in Part 1	Worked example 2
4.1.2	B.4.3 Panel 35 Panel 35 contains several ancillaries in the form of ladders and feeder trays, etc. and hence the panel resistances cannot be derived in accordance with 4.2 of Part 1.
	NOTE The ancillaries are not reasonably identical on each face, and hence cannot be treated as structural members in accordance with 4.2 of Part 1.
4.1.3	However, approximate calculation of face areas shows the following.
	a) Maximum projected area of ancillaries in any face is less than 0.9 m^2; minimum area of structural members in any face is greater than 1.3 m^2, and hence: $\frac{\Sigma A_A}{\Sigma A_F} \leq \frac{0.9}{1.3} = 0.69 < 1.0$
	Therefore the panel complies with 4.1.3 a) of Part 1.
	b) Gross face area $\approx 10 \text{ m}^2$, and $\frac{A_{A \max}}{A_g} \leq \frac{0.9}{10} = 0.09 < 0.5$
	Therefore the panel complies with 4.1.3 b) of Part 1.
	c) No ancillaries extend outside the legs.
	Therefore the panel complies with 4.1.3 c) of Part 1.
4.3.1	Hence, the total resistance may be derived in accordance with 4.3 of Part 1 without recourse to the recommendations of 4.4 of Part 1.
	The total panel resistance, ΣR_w is given by:
	$\Sigma R_w = R_T + R_{AW}$
	The bare tower is composed of flat sections only, and is identical in each face. Its resistance, R_T, can thus be dealt with in an identical manner to panel 1 (see B.4.2).
	 The diagram shows a square tower section with a height of 3.05 m. The top width is 3.124 m and the bottom width is 3.350 m. The tower is composed of flat sections only. The members are labeled as follows: - Legs: $152 \times 152 \times 19 \text{ mm L}$ - Diagonals: $64 \times 64 \times 6 \text{ mm L}$ - Horizontals: $57 \times 57 \times 6 \text{ mm L}$ - Verticals: $57 \times 57 \times 6 \text{ mm L}$
4.2.1 a)	$A_s = A_f = (0.152 \times 3.05 \times 2) + (4.445 \times 0.064 \times 2) + (3.124 \times 0.057) + (1.524 \times 0.057) = 1.76 \text{ m}^2$
Figure 4.1	$\phi = \frac{2A_s}{h(b_1 + b_2)} = \frac{2 \times 1.76}{3.05(3.12 + 3.35)} = 0.18$ i.e. that appropriate to the bare tower.
	The wind incidence factor, K_θ, is derived from Figure 4.2 of Part 1 for $A_f/A_F = 1.0$ and $\phi = 0.18$:
Figure 4.2	For wind on face ($\theta = 0^\circ$), $K_\theta = 1.0$;
	For wind on diagonal ($\theta = 45^\circ$), $K_\theta = 1.11$.

Reference in Part 1	Worked example 2
Figure 4.3 a)	<i>Drag coefficients</i> $A_s A_f = A_F$ For $\phi = 0.18$, $C_N = C_{Nf} = 2.95$: $R_T = K_\theta C_N A_s$
4.2.1 a)	For $\theta = 0^\circ$: $R_T = 1.0 \times 2.95 \times 1.76 = 5.19 \text{ m}^2$
4.2.1 b)	For $\theta = 45^\circ$: $R_T = 1.11 \times 2.95 \times 1.76 = 5.76 \text{ m}^2$ The ancillaries are all linear, comprising a ladder, feeders and a cable tray. Hence their wind resistance may be derived in accordance with 4.5 of Part 1.
Figure 4.1	Plan view Ladder Feeders, trays, etc.
4.5	The resistance is evaluated in accordance with 4.5 of Part 1, i.e. $R_{AW} = C_N K_A A_A \sin^2 \psi$ where $K_A = K_A' \left\{ 1 - \frac{c}{2d} + \frac{2}{3} \left(\frac{c}{d} \right)^2 \right\}$
Table 4.2	The following items are constant for all faces: $K_A' = 0.6$ for ancillaries internal to the section for a square tower; $d = 3.24 \text{ m}$ (mean value at mid-height); $\sin^2 \psi \approx 1.0$ since all ancillaries are close to vertical. All circular items are subcritical since the Reynolds number is less than the critical value.
Table 4.1	$R_e = 1.5 \frac{\bar{V}_z D}{\nu} = \frac{1.5 \times 32.0 \times 0.05}{1.46 \times 10^{-5}} = 1.64 \times 10^5 < 2 \times 10^5$ Using $A_{A(f)}$ for flat items and $A_{A(c)}$ for circular items: Ladder on faces 1, 3: $A_{A(f)} = (3.05 \times 0.050) = 0.153 \text{ m}^2$ Faces 2, 4: $A_{A(f)} = (3.05 \times 0.006) \times 2 = 0.037 \text{ m}^2$ $A_{A(c)} = \left(\frac{3.05}{0.254} \right) \times 0.016 \times 0.254 = 0.049 \text{ m}^2$

Reference in Part 1	Worked example 2																																																																						
Table 4.1	Feeders, trays on faces 1,3: $A_{A(f)} = 0.150 \times 3.050 = 0.458 \text{ m}^2$ $A_{A(c)} = 0.050 \times 3.050 \times 2 = 0.305 \text{ m}^2$ Faces 2, 4: $A_{A(c)} = 0.050 \times 3.050 = 0.153 \text{ m}^2$ <i>Drag coefficients</i> The drag coefficients are taken from Table 4.1 of Part 1: for ladder and cable trays $C_{N(f)} = 2.0$; for feeders (and ladder rungs) $C_{N(c)} = 1.2$. This allows for cables being of type b), c) or d) in Table 4.1 of Part 1. The equivalent C_{NA} is given by: $C_{NA} = C_{N(f)}A_{A(f)} + C_{N(c)}A_{A(c)}$ Hence the total resistance of ancillaries for wind normal to each face can be tabulated as follows.																																																																						
	<table><tr><th>Face</th><th>Ancillary</th><th>c</th><th>C_{NA}</th><th>K_A</th><th>R_{AW}</th></tr><tr><td rowspan="3">1</td><td>Ladder</td><td>2.49</td><td>0.31</td><td>0.61</td><td>m² 0.19</td></tr><tr><td>Feeders (and trays)</td><td>3.14</td><td>1.28</td><td>0.69</td><td>0.89</td></tr><tr><td></td><td></td><td></td><td></td><td><u>1.08</u></td></tr><tr><td rowspan="3">2</td><td>Ladder</td><td>0.13</td><td>0.133</td><td>0.59</td><td>0.08</td></tr><tr><td>Feeders (and trays)</td><td>0.50</td><td>0.184</td><td>0.56</td><td>0.10</td></tr><tr><td></td><td></td><td></td><td></td><td><u>0.18</u></td></tr><tr><td rowspan="3">3</td><td>Ladder</td><td>0.50</td><td>0.31</td><td>0.56</td><td>0.17</td></tr><tr><td>Feeders (and trays)</td><td>0.05</td><td>1.28</td><td>0.60</td><td>0.76</td></tr><tr><td></td><td></td><td></td><td></td><td><u>0.93</u></td></tr><tr><td rowspan="3">4</td><td>Ladder</td><td>3.06</td><td>0.133</td><td>0.68</td><td>0.09</td></tr><tr><td>Feeders (and trays)</td><td>2.54</td><td>0.184</td><td>0.61</td><td>0.12</td></tr><tr><td></td><td></td><td></td><td></td><td><u>0.21</u></td></tr></table>	Face	Ancillary	c	C_{NA}	K_A	R_{AW}	1	Ladder	2.49	0.31	0.61	m ² 0.19	Feeders (and trays)	3.14	1.28	0.69	0.89					<u>1.08</u>	2	Ladder	0.13	0.133	0.59	0.08	Feeders (and trays)	0.50	0.184	0.56	0.10					<u>0.18</u>	3	Ladder	0.50	0.31	0.56	0.17	Feeders (and trays)	0.05	1.28	0.60	0.76					<u>0.93</u>	4	Ladder	3.06	0.133	0.68	0.09	Feeders (and trays)	2.54	0.184	0.61	0.12					<u>0.21</u>
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The resulting total resistance of the tower normal to each face is then:																																																																							
<table><tr><th>Face</th><th>R_T</th><th>R_{AW}</th><th>ΣR_W</th></tr><tr><td>1</td><td>5.19</td><td>1.08</td><td>m² 6.27</td></tr><tr><td>2</td><td>5.19</td><td>0.18</td><td>5.37</td></tr><tr><td>3</td><td>5.19</td><td>0.93</td><td>6.12</td></tr><tr><td>4</td><td>5.19</td><td>0.21</td><td>5.40</td></tr></table>	Face	R_T	R_{AW}	ΣR_W	1	5.19	1.08	m ² 6.27	2	5.19	0.18	5.37	3	5.19	0.93	6.12	4	5.19	0.21	5.40																																																			
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4.3.2																																																																							

Reference in Part 1	Worked example 2																			
	The resistance needs to be calculated for wind incidence at 45° to a face. The calculations for wind into leg A have been given here only (this direction will give the highest resistance).  K_A' and C_N are unchanged from the normal to face calculation. Projected area of ladder: $A_{A(f)} = 0.242 \text{ m}^2$; $A_{A(c)} = 0.035 \text{ m}^2$. Projected area of feeders and tray: $A_{A(f)} = 0.388 \text{ m}^2$ $A_{A(c)} = 0.305 \text{ m}^2$. <table><tr><th>Ancillary</th><th>c/d</th><th>$C_N A_A$</th><th>$K_A \sin^2 \psi$</th><th>R_{AW}</th></tr><tr><td rowspan="2">Ladder</td><td>$\frac{3.70}{3.88}$</td><td rowspan="2">0.53</td><td rowspan="2">0.68</td><td>m^2</td></tr><tr><td></td><td>0.36</td></tr><tr><td rowspan="2">Feeders and trays</td><td>$\frac{3.89}{3.96}$</td><td rowspan="2">1.14</td><td rowspan="2">0.69</td><td>0.79</td></tr><tr><td></td><td><u>1.15</u></td></tr></table> Wind resistance at 45° to faces 1 and 2 is then: $\Sigma R_w = 5.76 + 1.15 = \underline{6.91} \text{ m}^2$ NOTE This calculation should be repeated for each diagonal wind as required.	Ancillary	c/d	$C_N A_A$	$K_A \sin^2 \psi$	R_{AW}	Ladder	$\frac{3.70}{3.88}$	0.53	0.68	m^2		0.36	Feeders and trays	$\frac{3.89}{3.96}$	1.14	0.69	0.79		<u>1.15</u>
Ancillary	c/d	$C_N A_A$	$K_A \sin^2 \psi$	R_{AW}																
Ladder	$\frac{3.70}{3.88}$	0.53	0.68	m^2																
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				<u>1.15</u>																

Reference in Part 1	Worked example 2																				
	B.4.4 Panel 46																				
4.1.2	Panel 46 contains several different types of ancillaries, such as dish aerials, platforms, ladders and feeders, hence the resistance of the panel cannot be derived in accordance with 4.2 of Part 1.																				
4.1.3	A quick check of the dimensions of the dish aerial reveals that it protrudes almost 4 m beyond the tower, i.e.: $\frac{\text{length of aerial past tower}}{\text{width of tower}} = \left(\frac{4}{14} \right) \approx 30 \% > 10 \%$																				
4.1.4	This does not comply with 4.1.3 c) of Part 1, and the resistance is derived in accordance with 4.4 of Part 1.																				
4.4.1	The total resistance is built up from the effective individual face resistances, R_1 to R_4 , which are derived allowing for the total combined resistance of the bare tower and all ancillaries and discrete attachments in each face of the tower.																				
4.4.2	a) <i>Area of structural members.</i> Since the tower is symmetrical, and all members are flat sided, for all faces: $A_s = A_f$; $C_n = C_{nf}$ A_f can be determined in a similar way to panel 35 (see B.4.3) for all members on the face, i.e. excluding all hip and plan bracing, giving: $A_s = 13.04 \text{ m}^2$ C_{nf} for the tower face cannot be determined until the areas of ancillaries are calculated, and the consequent solidity ratio for each face determined.																				
4.6	b) <i>Areas of ancillaries and attachments</i> 1) <i>Dish aerial</i> at 24.4 m (faces 1 and 4). Data provided by supplier (or wind tunnel tests): $A_A = 4.71 \text{ m}^2$ and $C_n = 1.46$. This is situated on leg A, and to allow possible directional orientation is taken as applying equally to face 1 and face 4. 2) <i>Platforms and ladder</i> (faces 1 and 4). These are all virtually symmetrical about leg A, and can be summed together to give areas appropriate to both faces 1 and 4, and ϕ_A relevant to each attachment.																				
	<table><tr><th>Item</th><th>$A_{A(f)}$</th><th>$A_{A(c)}$</th><th>ϕ_A</th></tr><tr><td>(i) Platform at 24.4 m</td><td>1.36</td><td>0.69</td><td>0.36</td></tr><tr><td>(ii) Platform at 21.3 m</td><td>1.10</td><td>0.44</td><td>0.32</td></tr><tr><td>(iii) Ladder and other items</td><td>2.11</td><td>0.13</td><td>0.24</td></tr><tr><td></td><td>4.57</td><td>1.26</td><td></td></tr></table> $\Sigma A_A = \underline{5.83} \text{ m}^2$	Item	$A_{A(f)}$	$A_{A(c)}$	ϕ_A	(i) Platform at 24.4 m	1.36	0.69	0.36	(ii) Platform at 21.3 m	1.10	0.44	0.32	(iii) Ladder and other items	2.11	0.13	0.24		4.57	1.26	
Item	$A_{A(f)}$	$A_{A(c)}$	ϕ_A																		
(i) Platform at 24.4 m	1.36	0.69	0.36																		
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(iii) Ladder and other items	2.11	0.13	0.24																		
	4.57	1.26																			

Reference in Part 1	Worked example 2												
4.5	The diameters of all circular elements are not large enough for supercritical regime. Hence from 4.4.2 of Part 1, the corresponding C_n are:												
Figure 4.5	<table><tr><th>Item</th><th>C_{nf}</th><th>C_{nc}</th></tr><tr><td>(i)</td><td>1.65</td><td>1.04</td></tr><tr><td>(ii)</td><td>1.68</td><td>1.04</td></tr><tr><td>(iii)</td><td>1.75</td><td>1.06</td></tr></table>	Item	C_{nf}	C_{nc}	(i)	1.65	1.04	(ii)	1.68	1.04	(iii)	1.75	1.06
Item	C_{nf}	C_{nc}											
(i)	1.65	1.04											
(ii)	1.68	1.04											
(iii)	1.75	1.06											
4.4.2	Hence equivalent C_n $= \frac{1}{5.83} \left((1.65 \times 1.36) + (1.68 \times 1.10) + (1.75 \times 2.11) + (1.05 \times 1.26) \right)$ $= \underline{1.56}$ $\uparrow$ $\left[\begin{array}{c} \text{average} \\ C_{nc} \end{array} \right]$												
Table 4.1	3) <i>Feeders and trays</i> (faces 2 and 3) Face 2: $A_{A(c)} = \underline{0.61} \text{ m}^2$; $C_n = 1.2$ Face 3: $A_{A(c)} = \underline{1.22} \text{ m}^2$; $C_n = 1.2$ $A_{A(f)} = \underline{1.83} \text{ m}^2$; $C_n = 2.0$ $\Sigma A_A = \underline{3.05} \text{ m}^2$ c) <i>Solidity ratios</i>. Solidity ratios are based on all elements within a face, with due allowance in the enveloping area for those items protruding significantly past the face width:												
Figure 4.1	$\phi = \frac{A_s + \Sigma A_A}{h \times b_e}$ where b_e is an equivalent face width to cater for dish projections, etc. where appropriate e.g.: 												

Reference in Part 1	Worked example 2																																			
	<table><tr><th>Face</th><th>A_S m²</th><th>ΣA_A m²</th><th>h m</th><th>b_e m</th><th>ϕ</th><th>C_{nf}</th></tr><tr><td>1</td><td>13.04</td><td>10.54</td><td>12.2</td><td>16.95</td><td>0.12</td><td>1.83</td></tr><tr><td>2</td><td>13.04</td><td>0.61</td><td>12.2</td><td>15.15</td><td>0.07</td><td>1.93</td></tr><tr><td>3</td><td>13.04</td><td>3.05</td><td>12.2</td><td>15.15</td><td>0.09</td><td>1.88</td></tr><tr><td>4</td><td>13.04</td><td>10.54</td><td>12.2</td><td>16.95</td><td>0.12</td><td>1.83</td></tr></table>	Face	A_S m ²	ΣA_A m ²	h m	b_e m	ϕ	C_{nf}	1	13.04	10.54	12.2	16.95	0.12	1.83	2	13.04	0.61	12.2	15.15	0.07	1.93	3	13.04	3.05	12.2	15.15	0.09	1.88	4	13.04	10.54	12.2	16.95	0.12	1.83
Face	A_S m ²	ΣA_A m ²	h m	b_e m	ϕ	C_{nf}																														
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4.4.2 Figure 4.5	d) <i>Total face resistance</i> The total face resistance can now be obtained using ϕ from c) for tower drag coefficient: $C_n = C_{nf}$																																			
4.4.1	and $R = C_n A_s + \Sigma (C_{nA} A_A)$ $R_1 = (1.83 \times 13.04) + (1.46 \times 4.71) + (1.56 \times 5.83)$ $= 23.9 + 6.9 + 9.0 = R_4 = \underline{39.8} \text{ m}^2$ $R_2 = (1.93 \times 13.04) + (1.2 \times 0.61) = \underline{25.9} \text{ m}^2$ $R_3 = (1.88 \times 13.04) + (1.2 \times 1.22) + (2.0 \times 1.83)$ $= 24.5 + 1.5 + 3.6 = \underline{29.6} \text{ m}^2$																																			
4.4.1 Figure 4.2	e) <i>Wind resistance according to direction</i> For wind normal to face, $\theta = 0^\circ$ $\Sigma R_w = R_e = (R_F + \eta_F R_B)$, since $K_\theta = 1$ For wind on corner, $\theta = 45^\circ$, and using a mean K_θ $\Sigma R_w = \epsilon K_\theta (R_{F1} + R_{F2} + \eta_{F1} R_{B1} + \eta_{F2} R_{B2})$ where suffixes F and B refer to front and back faces, respectively. From Figure 4.2 of Part 1, we have averaged K_θ thus:																																			
Figure 4.2	<table><tr><th>Corner</th><th>ΣA_f m²</th><th>ΣA_F m²</th><th>$\Sigma A_f / \Sigma A_F$</th><th>ϕ_{AV}</th><th>K_θ</th></tr><tr><td>1/2</td><td>26.08</td><td>37.23</td><td>0.70</td><td>0.10</td><td>1.12</td></tr><tr><td>2/3</td><td>26.08</td><td>29.74</td><td>0.88</td><td>0.08</td><td>1.11</td></tr><tr><td>3/4</td><td>26.08</td><td>39.67</td><td>0.66</td><td>0.11</td><td>1.12</td></tr><tr><td>4/1</td><td>26.08</td><td>47.16</td><td>0.55</td><td>0.12</td><td>1.14</td></tr></table>	Corner	ΣA_f m ²	ΣA_F m ²	$\Sigma A_f / \Sigma A_F$	ϕ_{AV}	K_θ	1/2	26.08	37.23	0.70	0.10	1.12	2/3	26.08	29.74	0.88	0.08	1.11	3/4	26.08	39.67	0.66	0.11	1.12	4/1	26.08	47.16	0.55	0.12	1.14					
Corner	ΣA_f m ²	ΣA_F m ²	$\Sigma A_f / \Sigma A_F$	ϕ_{AV}	K_θ																															
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3/4	26.08	39.67	0.66	0.11	1.12																															
4/1	26.08	47.16	0.55	0.12	1.14																															
	NOTE K_θ values here are very similar due to the low values of ϕ; this will <i>not</i> be the case when $\phi \geq 0.2$ (see Figure 4.2 of Part 1). For this panel, for all corner wind directions, $0.5 K_\theta$ could be taken as 0.57, but more generally, the value according to each corner should be used.																																			
	f) <i>Shielding factors</i>. Now $A_c = A_c' = 0$ (all structural members are flat sided). Then $\eta' = \eta_f \left(\frac{A_f + A_A}{A_F} \right) = \eta_f$ Also for square-section towers $\omega = 1$ Therefore $\eta = \eta' = \eta_f$, giving: faces 1 and 4: $\phi = 0.12$; $\eta = 0.78$; face 2: $\phi = 0.07$; $\eta = 0.87$; face 3: $\phi = 0.09$; $\eta = 0.83$.																																			

Reference in Part 1	Worked example 2																																																																																																										
4.2 4.3 4.4 4.4 5.3.1	g) <i>Total panels resistances</i> The total panel resistance can now be determined using the data in a) to f): <table><tr><th>Wind direction on to</th><th>R_F</th><th>R_B</th><th>ηF</th><th>K_θ</th><th>ΣR_W</th></tr><tr><td></td><td>m²</td><td>m²</td><td></td><td></td><td>m²</td></tr><tr><td>Face 1</td><td>39.8</td><td>29.6</td><td>0.78</td><td>1.0</td><td>62.9</td></tr><tr><td>Face 2</td><td>25.9</td><td>39.8</td><td>0.87</td><td>1.0</td><td>60.5</td></tr><tr><td>Face 3</td><td>29.6</td><td>39.8</td><td>0.83</td><td>1.0</td><td>62.6</td></tr><tr><td>Face 4</td><td>39.8</td><td>25.9</td><td>0.78</td><td>1.0</td><td>60.0</td></tr><tr><td></td><td colspan="2">$\Sigma R_F + \Sigma (R_B \eta_F)$</td><td colspan="2">$0.5 K_\theta$</td><td>$\Sigma R_W$</td></tr><tr><td>Corner 1/2</td><td colspan="2">$\equiv (62.9 + 60.5)$</td><td colspan="2">0.56</td><td>69.1</td></tr><tr><td>Corner 2/3</td><td colspan="2">$\equiv (60.5 + 62.6)$</td><td colspan="2">0.55</td><td>67.7</td></tr><tr><td>Corner 3/4</td><td colspan="2">$\equiv (62.6 + 60.0)$</td><td colspan="2">0.56</td><td>68.7</td></tr><tr><td>Corner 4/1</td><td colspan="2">$\equiv (60.0 + 62.9)$</td><td colspan="2">0.57</td><td>70.1</td></tr></table> NOTE These resistances do not differ greatly between directions, but in cases where $\phi > 0.1$ and $\Sigma A_A \geq A_F$, there will be greater differences. B.4.5 General summary (resistance) In a similar way to the methods detailed in B.4.2 to B.4.4, all the panel resistances can be determined in accordance with 4.2, 4.3 or 4.4 of Part 1, as required. For all panels with resistances determined in accordance with 4.4 of Part 1, as well as ΣR_W, the contribution due to ancillaries and cables will be required, ($\Sigma R_{AW} + \Sigma R_c$), since these are needed for determining loads on the “equivalent bare tower” in deriving fluctuating loads and gust response factors in accordance with B.5. Thus for panel 46, $\Sigma R_c = 0$ and $\Sigma R_{AW} = \Sigma (C_{nA} A_A)$ see B.4.4 d). <table><tr><th>Wind direction</th><th>ΣR_W</th><th>ΣR_{AW}</th><th>R_{TE}</th></tr><tr><td></td><td>m²</td><td>m²</td><td>m²</td></tr><tr><td>Face 1</td><td>62.9</td><td>15.9 + 5.1</td><td>41.9</td></tr><tr><td>Face 2</td><td>60.5</td><td>0.7 + 15.9</td><td>43.9</td></tr><tr><td>Face 3</td><td>62.6</td><td>5.1 + 15.9</td><td>41.6</td></tr><tr><td>Face 4</td><td>60.0</td><td>15.9 + 0.7</td><td>43.4</td></tr><tr><td>Corner 1/2</td><td>69.1</td><td>6.9 + (1.41 × 14.1)</td><td>42.5</td></tr><tr><td>Corner 2/3</td><td>67.7</td><td>(9.7 × 1.41) + 6.9</td><td>47.2</td></tr><tr><td>Corner 3/4</td><td>68.7</td><td>(14.1 × 1.41) + 6.9</td><td>42.1</td></tr><tr><td>Corner 4/1</td><td>70.1</td><td>6.9 + (1.41 × 9.7)</td><td>49.6</td></tr></table> NOTE An approximate check on the tower-only resistance can be derived from: $R = A_s C_n (1 + \eta)$ which using typical face values from the above gives: $R \approx 13.04 \times 1.9 \times (1 + 0.8) = 44.6 \text{ m}^2.$	Wind direction on to	R_F	R_B	ηF	K_θ	ΣR_W		m ²	m ²			m ²	Face 1	39.8	29.6	0.78	1.0	62.9	Face 2	25.9	39.8	0.87	1.0	60.5	Face 3	29.6	39.8	0.83	1.0	62.6	Face 4	39.8	25.9	0.78	1.0	60.0		$\Sigma R_F + \Sigma (R_B \eta_F)$		$0.5 K_\theta$		ΣR_W	Corner 1/2	$\equiv (62.9 + 60.5)$		0.56		69.1	Corner 2/3	$\equiv (60.5 + 62.6)$		0.55		67.7	Corner 3/4	$\equiv (62.6 + 60.0)$		0.56		68.7	Corner 4/1	$\equiv (60.0 + 62.9)$		0.57		70.1	Wind direction	ΣR_W	ΣR_{AW}	R_{TE}		m ²	m ²	m ²	Face 1	62.9	15.9 + 5.1	41.9	Face 2	60.5	0.7 + 15.9	43.9	Face 3	62.6	5.1 + 15.9	41.6	Face 4	60.0	15.9 + 0.7	43.4	Corner 1/2	69.1	6.9 + (1.41 × 14.1)	42.5	Corner 2/3	67.7	(9.7 × 1.41) + 6.9	47.2	Corner 3/4	68.7	(14.1 × 1.41) + 6.9	42.1	Corner 4/1	70.1	6.9 + (1.41 × 9.7)	49.6
	Wind direction on to	R_F	R_B	ηF	K_θ	ΣR_W																																																																																																					
		m ²	m ²			m ²																																																																																																					
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		$\Sigma R_F + \Sigma (R_B \eta_F)$		$0.5 K_\theta$		ΣR_W																																																																																																					
	Corner 1/2	$\equiv (62.9 + 60.5)$		0.56		69.1																																																																																																					
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Reference in Part 1	Worked example 2
5.1 5.1.1	B.5 Structural response to wind B.5.1 General a) <i>Check for use of the static method.</i> The equivalent static method may be used to assess the maximum member forces in the tower if: $\rho_s \frac{7 m_T}{R_{WT} \sqrt{(d_B \tau_o)}} \left(\frac{5}{6} - \frac{h_T}{H} \right)^2 < 1$ The minimum total tower resistance including ancillaries: $\Sigma R_W = 433 \text{ m}^2$ Therefore $\frac{\Sigma R_W}{3} = 144 \text{ m}^2$ The panel resistance summation just less than this, corresponds to panels 1 to 40 inclusive, for which: $R_{WT} = 141.2 \text{ m}^2$ The height to the underside of panel 40 = 65 m. Therefore, the height of panels 1 to 40 = $135 - 65 = 70 \text{ m} = h'_t$ $h_t = 70 \text{ or } H/3, \text{ whichever is less}$ $= 70 \text{ m or } 45 \text{ m}$ Therefore $h_t = 45 \text{ m};$ $H = 135 \text{ m}$ $d_B = 21.3 \text{ m};$ $\tau_o = 0.001 \text{ m};$ $\rho_s = 7\,850 \text{ kg/m}^3$ $m_T = 28\,870 \text{ kg, including all ancillaries for panels 1 to 40.}$ Therefore $\frac{7 \times 28870}{7850 \times 141.2} \frac{1}{\sqrt{(0.001 \times 21.3)}} \left(\frac{5}{6} - \frac{1}{3} \right)^2 = 0.31 < 1$ Therefore the equivalent static method may be used. b) <i>Procedure to be used.</i> Although panels 1 and 35 (and many other panels) comply with the constraints of 4.1.3 of Part 1, the forces cannot be evaluated using 5.2 of Part 1, since several panels do not comply with those recommendations (e.g. panel 46). Hence the wind loading for analysis and the member forces so derived have to be determined in accordance with 5.3 of Part 1. c) <i>Mean wind loads</i> <i>Panel 1</i> For wind blowing normal to each face: for $H = 135 \text{ m}$: $\bar{V}_z = 19.8 \times (12.5)^{0.23} = 35.4 \text{ m/s}$ $R_{TE} = \Sigma R_W - (R_{AW} + R_{CW}) = 1.16 - 0 = 1.16$ Therefore $\bar{P}_{TW} = \frac{1.22}{2} \times \frac{35.4^2}{1000} \times 1.16 = 0.89 \text{ kN}$
5.1.2	b) <i>Procedure to be used.</i> Although panels 1 and 35 (and many other panels) comply with the constraints of 4.1.3 of Part 1, the forces cannot be evaluated using 5.2 of Part 1, since several panels do not comply with those recommendations (e.g. panel 46). Hence the wind loading for analysis and the member forces so derived have to be determined in accordance with 5.3 of Part 1.
5.3.1	c) <i>Mean wind loads</i>
3.2.1	<i>Panel 1</i>
5.2.1	For wind blowing normal to each face:

Reference in Part 1	Worked example 2																																																		
5.2.1	Similarly, for wind on any diagonal: $\bar{P}_{TW} = 0.89 \times \frac{1.39}{1.16} = \underline{1.06} \text{ kN}$ <i>Panel 35</i> The method of calculating the loads with resistances determined in accordance with 4.3 of Part 1 is similar to that for panel 1, but the total loads will differ for each face direction:																																																		
	3.2.1for H = 91 m: $\bar{V}_z = 19.8 \times (8.1)^{0.23} = \underline{32.0} \text{ m/s}$ Since $R_{TE} \equiv \Sigma R_W$, the loads can be tabulated thus, using: $\bar{P}_{TW} = \frac{1}{2} \rho_a \bar{V}_z^2 \times R_{TE} = 0.625 R_{TE} \text{ (kN)}$																																																		
	4.3.2 a)NOTE The R_{AW} term used in determining ΣR_W in accordance with 4.3 of Part 1, is not subtracted, since ΣR_W is an “effective total tower resistance”, R_{TE}																																																		
	5.3 <table><tr><th>Direction</th><th>R_{TE}</th><th>$\bar{P}_{TW}$</th></tr><tr><td></td><td>m²</td><td>kN</td></tr><tr><td>Face 1</td><td>6.27</td><td>3.92</td></tr><tr><td>Face 2</td><td>5.37</td><td>3.35</td></tr><tr><td>Face 3</td><td>6.12</td><td>3.82</td></tr><tr><td>Face 4</td><td>5.40</td><td>3.37</td></tr><tr><td>Diagonal 1/4</td><td>6.91</td><td>4.32</td></tr></table>	Direction	R_{TE}	$\bar{P}_{TW}$		m ²	kN	Face 1	6.27	3.92	Face 2	5.37	3.35	Face 3	6.12	3.82	Face 4	5.40	3.37	Diagonal 1/4	6.91	4.32																													
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Face 4	5.40	3.37																																																	
Diagonal 1/4	6.91	4.32																																																	
3.2.1	<i>Panel 46</i> Again the loads can be set down in a similar form to panel 35, but with separate items for $\bar{P}_{TE}$ and $\bar{P}_{AW}$ as follows: $H = 19.6 \text{ m} < 20 \text{ m}$ $\bar{V}_z = 9.9 \left(1 + \frac{19.6}{20} \right) = 19.6 \text{ m/s}$																																																		
5.2.1 and 5.3.2	$\bar{P} = \frac{1}{2} \rho_a \bar{V}_z^2 \times R = 0.234 \times R \text{ (kN)}$ <table><tr><th>Direction</th><th>R_{TE}</th><th>ΣR_{AW}</th><th>$\bar{P}_{TE}$</th><th>$\bar{P}_{AW}$</th></tr><tr><td></td><td>m²</td><td>m²</td><td>kN</td><td>kN</td></tr><tr><td>Face 1</td><td>41.9</td><td>21.0</td><td>9.81</td><td>4.91</td></tr><tr><td>Face 2</td><td>43.9</td><td>16.6</td><td>10.23</td><td>3.88</td></tr><tr><td>Face 3</td><td>41.6</td><td>21.0</td><td>9.73</td><td>4.91</td></tr><tr><td>Face 4</td><td>43.4</td><td>16.6</td><td>10.16</td><td>3.88</td></tr><tr><td>Corner 1/2</td><td>42.5</td><td>26.6</td><td>9.95</td><td>6.22</td></tr><tr><td>Corner 2/3</td><td>47.2</td><td>20.5</td><td>11.05</td><td>4.80</td></tr><tr><td>Corner 3/4</td><td>42.1</td><td>26.6</td><td>9.85</td><td>6.22</td></tr><tr><td>Corner 4/1</td><td>49.6</td><td>20.5</td><td>11.61</td><td>4.80</td></tr></table> NOTE The partition of loads between $\bar{P}_{TE}$ and $\bar{P}_{AW}$ is not crucial and any reasonable assumption for the partition of ΣR can be made, provided: 1) ΣR_{AW} is any upper bound to the total resistance of all discrete attachments excluding shielding effects; 2) $R_{TE} + \Sigma R_{AW} = \Sigma R_W$.	Direction	R_{TE}	ΣR_{AW}	$\bar{P}_{TE}$	$\bar{P}_{AW}$		m ²	m ²	kN	kN	Face 1	41.9	21.0	9.81	4.91	Face 2	43.9	16.6	10.23	3.88	Face 3	41.6	21.0	9.73	4.91	Face 4	43.4	16.6	10.16	3.88	Corner 1/2	42.5	26.6	9.95	6.22	Corner 2/3	47.2	20.5	11.05	4.80	Corner 3/4	42.1	26.6	9.85	6.22	Corner 4/1	49.6	20.5	11.61	4.80
Direction	R_{TE}	ΣR_{AW}	$\bar{P}_{TE}$	$\bar{P}_{AW}$																																															
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5.3.1																																																			

Reference in Part 1	Worked example 2																																																	
5.2.2	d) <i>Gust response factors</i>																																																	
5.2.3	1) <i>For tower loads.</i> For legs: $G_1 = B j \left\{ 1 + 0.2 \left(\frac{z}{H} \right)^2 \right\}$																																																	
5.2.4	For bracings: $G_b = K_q G_1$ where K_q is derived from the mean hourly analyses on the partially shielded tower body, using Figure 5.4 of Part 1 according to the ratio $1/f_q$, where f_q is the proportion of the shear carried by the bracings to the total at the level considered.																																																	
Figure 5.4	These are required for each level up the tower, but are the same regardless of direction. For lower sections of the tower with complex bracing arrangements, it may be easier to determine the proportion of shear carried by the bracing system by resolving the leg member forces under mean wind on the effective tower body and subtracting this from the total.																																																	
Figures 5.2, 5.3 and 5.4	Typical gust factors, for members in the panels considered above, are as follows:																																																	
5.2.4	<table><tr><th>Panel</th><th>z</th><th>$H - z$</th><th>z/H</th><th>B</th><th>j</th><th>G_B</th><th>f_q</th><th>K_q</th><th>G_1</th><th>G_b</th></tr><tr><td>1</td><td>135</td><td>(13.5)</td><td>1.00</td><td>3.19</td><td>0.53</td><td>1.69</td><td>1.00</td><td>1.00</td><td>2.03</td><td>2.03</td></tr><tr><td>35</td><td>91</td><td>44</td><td>0.67</td><td>2.30</td><td>0.72</td><td>1.66</td><td>0.42</td><td>1.17</td><td>1.80</td><td>2.10</td></tr><tr><td>46</td><td>20</td><td>115</td><td>0.15</td><td>1.72</td><td>0.96</td><td>1.65</td><td>0.11^a</td><td>2.72</td><td>1.66</td><td>4.51</td></tr></table> ^a $1/f_q = 9.1 (> 8)$, so $K_q = 1 + \frac{115^{0.4}}{23.6} (9.1 - 3) = 2.72$	Panel	z	$H - z$	z/H	B	j	G_B	f_q	K_q	G_1	G_b	1	135	(13.5)	1.00	3.19	0.53	1.69	1.00	1.00	2.03	2.03	35	91	44	0.67	2.30	0.72	1.66	0.42	1.17	1.80	2.10	46	20	115	0.15	1.72	0.96	1.65	0.11 ^a	2.72	1.66	4.51					
Panel	z	$H - z$	z/H	B	j	G_B	f_q	K_q	G_1	G_b																																								
1	135	(13.5)	1.00	3.19	0.53	1.69	1.00	1.00	2.03	2.03																																								
35	91	44	0.67	2.30	0.72	1.66	0.42	1.17	1.80	2.10																																								
46	20	115	0.15	1.72	0.96	1.65	0.11 ^a	2.72	1.66	4.51																																								
5.2.2.3	NOTE 1 $\bar{G}_B$ from the “simple method” (see 5.2.2.3 of Part 1) gives $G_B = 1.65$.																																																	
5.2.2.2	NOTE 2 Figures in parentheses indicate use of minimum value of 0.1 H when $H > 100$ m; G_B applies to all levels above 121.5 m.																																																	
5.3.2	2) <i>For loads on attachments.</i> There are no cables, so 5.3.3 of Part 1 is not required and appropriate gust factors can be derived in accordance with 5.3.2 of Part 1, using: $G_A = B_A j_A [1 + 0.2 (Z_A/H)^2]$ Unlike G_B , these do not relate to the level of the members being considered, but solely to the size and level of the ancillary, and hence are determined for each major attachment as follows, using Figure 5.1 and Figure 5.2 of Part 1:																																																	
Figures 5.1 and 5.2	<table><tr><th>Item^a</th><th>z_A</th><th>e_A^b</th><th>e_A/Z_A^c</th><th>BA</th><th>j_A</th><th>G_A</th></tr><tr><td>1. 11 m long VHF</td><td>123</td><td>12.3</td><td>0.10</td><td>3.17</td><td>0.53</td><td>1.96</td></tr><tr><td>2. 2.4 m ϕ dish</td><td>110</td><td>11.0</td><td>0.10</td><td>3.22</td><td>0.53</td><td>1.93</td></tr><tr><td>3. 2.4 m ϕ dish</td><td>98</td><td>10.0</td><td>0.10</td><td>3.30</td><td>0.53</td><td>1.93</td></tr><tr><td>4. 14 m long VHF</td><td>78</td><td>14.0</td><td>0.18</td><td>3.08</td><td>0.62</td><td>2.04</td></tr><tr><td>5. 2.1 m ϕ dish</td><td>70</td><td>10.0</td><td>0.14</td><td>3.30</td><td>0.58</td><td>2.02</td></tr><tr><td>6. 2.4 m ϕ dish</td><td>24</td><td>10.0</td><td>0.42</td><td>3.30</td><td>0.77</td><td>2.56</td></tr></table> ^a Includes associated platforms, etc. ^b e_A should not be taken as less than 10 m or 0.1 z_A. ^c e_A/z_A should not be taken as less than $10/z_A$ or 0.1, i.e. value e_A/z_A.	Item ^a	z_A	e_A ^b	e_A/Z_A ^c	BA	j_A	G_A	1. 11 m long VHF	123	12.3	0.10	3.17	0.53	1.96	2. 2.4 m ϕ dish	110	11.0	0.10	3.22	0.53	1.93	3. 2.4 m ϕ dish	98	10.0	0.10	3.30	0.53	1.93	4. 14 m long VHF	78	14.0	0.18	3.08	0.62	2.04	5. 2.1 m ϕ dish	70	10.0	0.14	3.30	0.58	2.02	6. 2.4 m ϕ dish	24	10.0	0.42	3.30	0.77	2.56
Item ^a	z_A	e_A ^b	e_A/Z_A ^c	BA	j_A	G_A																																												
1. 11 m long VHF	123	12.3	0.10	3.17	0.53	1.96																																												
2. 2.4 m ϕ dish	110	11.0	0.10	3.22	0.53	1.93																																												
3. 2.4 m ϕ dish	98	10.0	0.10	3.30	0.53	1.93																																												
4. 14 m long VHF	78	14.0	0.18	3.08	0.62	2.04																																												
5. 2.1 m ϕ dish	70	10.0	0.14	3.30	0.58	2.02																																												
6. 2.4 m ϕ dish	24	10.0	0.42	3.30	0.77	2.56																																												
5.3.2																																																		

Reference in Part 1	Worked example 2
(C.5.3.4 of this Part of BS 8100)	According to the values of G_A, separate sets of mean hourly analyses are required for each ancillary item. However, as demonstrated here, for ancillaries of virtually any size, the value of G_A will generally depend on the level of attachment, and provided the resulting G_A values are similar, the number of analyses can be reduced by grouping together. Hence for the attachments, only three sets of analyses are required: i) items 1, 2 and 3, for which $G_{A1} \approx 1.95$ can be used; ii) items 4 and 5, for which $G_{A2} = 2.03$ can be used; iii) item 6, for which $G_{A3} = 2.56$ is used.
5.3.4 (C.5.3.4 of this Part of BS 8100) (C.5.3.4 of this Part of BS 8100)	B.5.2 Member forces Having undertaken the appropriate elastic analyses under mean hourly wind loading for the shielded tower and for each set of attachments, together with an analysis under dead load, the member forces can be obtained using 5.3.4 of Part 1 (see C.5.3.4). Thus in general terms; $\Sigma F_{\max} = F_{DL} + \Sigma \bar{F}_W + \Sigma F'_W \sqrt{1 + \left(\frac{\Sigma F'_x}{\Sigma F'_W} \right)^2}$ where $\Sigma \bar{F}_W = (\bar{F}_{TE} + \bar{F}_{A1} + \bar{F}_{A2} + \bar{F}_{A3})_{\max}$ $\Sigma F'_W = G \bar{F}_{TE} + G_{A1} \bar{F}_{A1} + G_{A2} \bar{F}_{A2} + G_{A3} \bar{F}_{A3}$ $\Sigma F'_x = \frac{1}{2} [G \bar{F}_{TEX} + G_{A1} \bar{F}_{A1X} + G_{A2} \bar{F}_{A2X} + G_{A3} \bar{F}_{A3X}]$ where suffix X indicates $\bar{F}$ due to mean hourly loads normal to the maximum down-wind direction. The simplified procedure discussed in C.5.3.4 may be used for this tower leading to: $\Sigma F_{\max} = F_{DL} + \bar{F}_{TE \max} (1 + 1.12 G) + \sum_{1 \text{ to } 3} \bar{F}_{An} (1 + 1.12 G_{An})$ $\simeq F_{DL} + (\bar{F}_{TE} + \sum_{1 \text{ to } 3} \bar{F}_{An})_{\max} (1 + 1.12 G)$ since at high levels on the tower $G_A \approx G$, and at lower levels $\Sigma \bar{F}_A \ll \Sigma \bar{F}_{TE}$ for this tower.
5.1.4 5.3.5 3.3.2 5.2.5	B.5.3 Deflections for serviceability checks The maximum mean hourly deflection, δ, should be obtained from each set of analyses. The required Δ for use in 3.3.2 of Part 1 is then obtained as follows: $\Delta_a = (1 + G_B) \delta_{TE} + \sum_{1 \text{ to } 3} (1 + G_{An}) \delta_{An} \text{ in accordance with 5.2.5 a) of Part 1}$ $\Delta_b = \delta_{TE} + \sum_{1 \text{ to } 3} \delta_{An} \text{ in accordance with 5.2.5 b) of Part 1}$
5.1.4	The serviceability check would then be carried out in the same manner as in Appendix A.
2.5	B.6 Safety assessment The maximum member loads are checked against the design strength thus to achieve the required reliability with respect to strength: $\Sigma F_{\max} \leq \frac{Q_k}{\gamma_m} \text{ for each member}$ where Q_k is obtained in accordance with DD 133, or equivalent. See A.6.1 of Appendix A for typical calculations.

Appendix C Background to the use of Part 1

C.1 General

C.1.1 *Introduction and philosophy*

The scope of Part 1 was specified in a brief prepared by the British Standards Code of Practice Committee for Lattice Towers and Masts. Part 1 is intended to provide a rational basis for estimating loading for free-standing towers of lattice construction with due allowance for dynamic response to gusty winds. Guyed masts and offshore-mounted lattice towers are not fully covered by BS 8100 at present, for the reasons discussed in C.1.2.

Part 1 is suited to worldwide application and is so written that use may be made of the best meteorological information available. In view of the common practice to allow competitive design for strength and in the absence of a structural design code for lattice towers written in limit state format, safety factors on design loadings and on characteristic values of strength are given. DD 133 [1.1] provides a means of deriving characteristic strengths of members fabricated, erected and maintained to good practice. By use of the partial factors contained in Part 1, appropriate design strengths can be derived for other less well-controlled structures. Such strengths may need to be assessed when dealing with the appraisal of existing structures, or in work overseas. In this way it has been possible to overcome the problem of choice of load factor which followed the publication of CP 3: Chapter V: Part 2 [1.2]. The probabilistic nature of the primary loadings demanded statistical models. Thus, limit state principles and the partial load factor method of design have been adopted.

C.1.2 *Application to guyed masts and offshore-mounted lattice towers*

C.1.2.1 Guyed masts. The provisions in Part 1 for assessing the meteorological parameters to be used in design (section three) and wind resistance (section four) are also applicable to the design of guyed masts. However, the gust response of free-standing towers in which the first mode of vibration predominates, is inapplicable to guyed masts where many modes can contribute to the peak loading. In addition, correlation of gusts over lengths between stays affects the peak forces in the members. For these reasons, the gust response factors given in section five of Part 1 are inappropriate for guyed masts and recourse should be made to published sources to determine their response. Theoretical guidance is given in a CIRIA publication [1.3] and recommendations for the design and analysis of guyed masts are contained in recommendations published by the International Association for Shell and Spatial Structures [1.4].

In addition, the safety factors set out in section two of Part 1 have been based on calibration studies of lattice towers only, and on the provisions of the strength Draft for Development DD 133 [1.1] which was developed for lattice tower type construction. For these reasons, the provisions of section two of Part 1 may be inappropriate for guyed mast design, although they may be used for general guidance purposes.

It is intended to extend the scope of Part 1 to incorporate the recommendations for guyed masts at a future date.

C.1.2.2 Offshore-mounted lattice towers. Lattice towers mounted on offshore platforms are not covered by Part 1 as far as the meteorological parameters (section three) and safety factors (section two) are concerned. The gust response factors are inappropriate for inclined flare booms, and for booms supported other than solely at their base. Interference effects from the platform itself have a major influence on the wind loading and no provisions are contained in Part 1 to assess these effects. The provisions of section five of Part 1 are also related to land terrain and the figures in Part 1 would need to be extended for offshore conditions. Guidance on how Part 1 could be extended for application to these structures has been developed for the Department of Energy [1.5], and it is hoped that Part 1 will be extended to incorporate the necessary amendments covering all of these aspects, at a future date.

C.1.3 *References in C.1*

- 1.1 BRITISH STANDARDS INSTITUTION. *Strength assessment of members for towers of lattice construction*, 1st ed., 1985, DD 133.
- 1.2 BRITISH STANDARDS INSTITUTION. *Code of basic data for the design of buildings: Chapter V Loading: Part 2 Wind loads*, 4th ed., 1972, CP 3: Chapter V: Part 2.
- 1.3 CONSTRUCTION INDUSTRY RESEARCH AND INFORMATION ASSOCIATION. *The modern design of wind sensitive structures*, 1971.
- 1.4 INTERNATIONAL ASSOCIATION FOR SHELL AND SPATIAL STRUCTURES, WORKING GROUP NO. 4. *Recommendations for the Analysis and Design of Guyed Masts*, 1981.