Loughborough University

Further Structural Analysis and Geotechnical Design
22CVC101

Semester 2 2023 | In-Person Exam Paper

This examination is to take place in-person at a central University venue under exam conditions. The standard length of time for this paper is **3 hours**.

You will not be able to leave the exam hall for the first 30 or final 15 minutes of your exam. Your invigilator will collect your exam paper when you have finished.

Help during the exam

Invigilators are not able to answer queries about the content of your exam paper. Instead, please make a note of your query in your answer script to be considered during the marking process.

If you feel unwell, please raise your hand so that an invigilator can assist you.

You may use a calculator for this exam. It must comply with the University's Calculator Policy for In-Person exams, in particular that it must not be able to transmit or receive information (e.g. mobile devices and smart watches are **not** allowed).

THIS PAPER COMPRISES **SECTION A** AND **SECTION B.**

Answer **TWO QUESTIONS in Section A.**
Answer **TWO QUESTIONS in Section B.**

Please use a separate answer book for each section. Print **SECTION A or SECTION B** on the front of the applicable answer books.

All questions carry equal marks.

Formula sheet is attached.

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SECTION A
(Answer **TWO QUESTIONS in Section A)**

1. a) Trees can be a major cause of change in soil volume for shallow foundations on particular types of soil.

   i) Detail the soil types that are susceptible to such volume change and explain the properties that affect that susceptibility.

   [2 marks]

   ii) Explain how the planting or removal of trees may lead to a change in soil volume.

   [3 marks]

   iii) Explain the steps you could take to prevent or accommodate the effects of trees on the foundation of a new house.

   [3 marks]

   b) A motorway cantilever road sign is founded 2m below ground level in a firm clay on a 3m by 3m foundation 2m thick. Load and soil details are below. Assess the ultimate limit state design stability of the sign foundation for EC7 design approach 1, case 1.

   Q_{gk} = 200kN M_{qkx} =200kNm M_{qky}= 300 kNm

   Cu_k = 75kN/m^2 C_k' = 3 kN/m^2 φ_k' = 25^o

   $\gamma_{clay\ k}$ = 20kN/m^3 $\gamma_{conc\ k}$ = 24kN/m^3

   Note $q = P/A \pm M_y/bd^2 \pm M_x/b^2d$

   [17 marks]

2. a) Explain the concept of pile negative skin friction, and how it is incorporated in EC7 design. In addition, what construction measures can you take to allow it to be ignored in design.

   [8 marks]

   b) A 4 by 4 pile group extends from 3m below ground level to 13m through a layered clay (Details on Table Q2). The piles are precast driven piles 100mm square at 1m c-c spacing.

   Calculate the design capacity of the system to EC7 Design approach 1, case 2. Assume a Pile group efficiency of 0.7, an Alpha of 0.35 and model factor 1.4.

Table Q2

Layer depth (m)	Φ (°)	γ (kN/m^3)	Cu (kN/m^2)
Ground level to 8m, clay	0	22	40
8m to 18m		20	Varies linearly 40 to 100

[17 marks]

Continues/…

3. In 1998 the A34 Newbury bypass resulted in the creation of a cut slope in London Clay (**Figure Q3a**). Laboratory tests carried out on samples of grey London Clay (γ_{sat}=20.2kN/m^2) delivered the results shown in **Figure Q3b.** It is assumed that the weathered London Clay (γ_{sat}=19.5kN/m^3) is approximately 10% weaker than the unweathered grey London Clay).

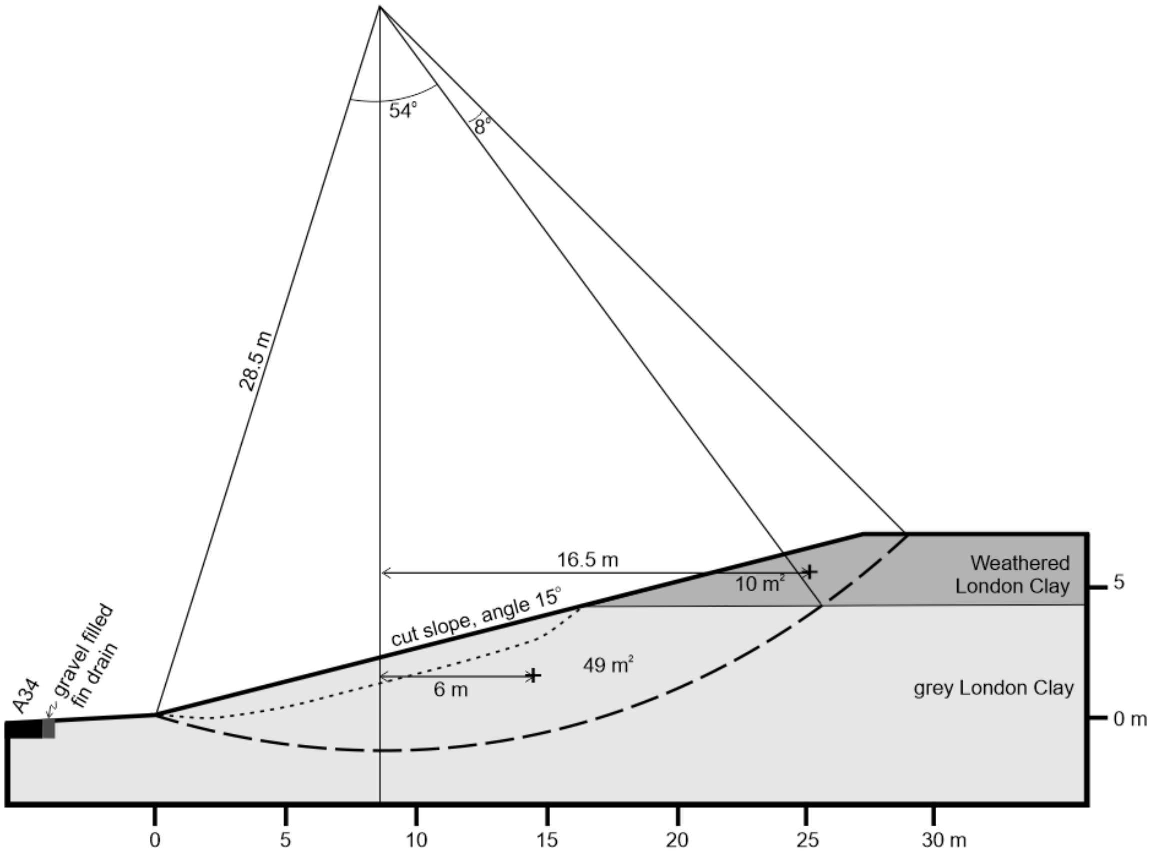

Figure Q3a

Question 3 continues/…

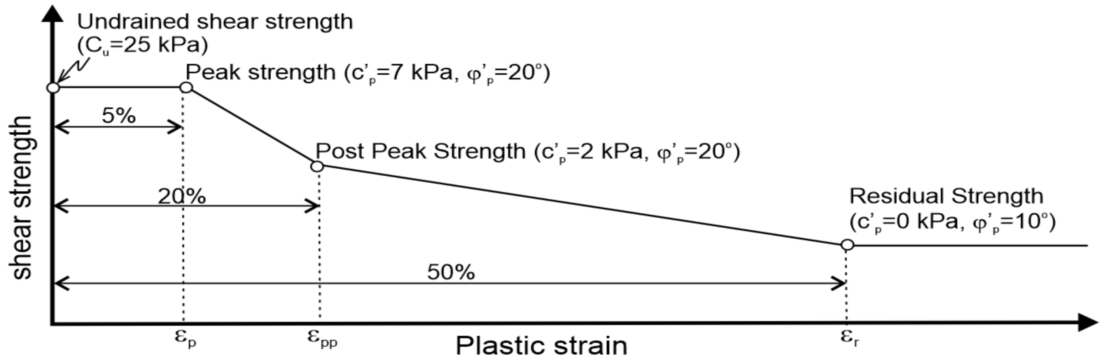

Figure Q3b

a) Determine the stability of this slope when it was just created in 1998. The slip circle characterising the most critical condition at this stage is indicated by the bold broken line in figure Q3a Clearly show your workings and list all conditions and assumptions relevant to your solution.

[8 marks]

b) The National Highways asset management team is interested in the long-term performance of this slope. Climate impact (weather cycles) drives deterioration of the slope material, particularly in a zone about 1 m below the ground surface along the central/lower section of the slope. The critical slip surface of the slope in its deteriorated state is indicated by the thin broken line in Figure Q3a.

Considering the deterioration in shear strength sketched in figure Q3b, evaluate at what stage the stability of this slope will become a matter for concern. Clearly show your workings, explain your choice of slope stability method and list all conditions and assumptions relevant to your solution.

[17 marks]

Continues/…

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SECTION B
(Answer **TWO QUESTIONS in Section B)**

4. a) The frame shown in Figure Q4 is to be analysed using the Stiffness Matrix method. The frame is pinned at joints 1 and 2 and is fixed at joint 3.

   i) Draw a diagram showing the restrained structure and the numbering system for the overall degrees of freedom.

   [3 marks]

   ii) Generate the overall stiffness matrix [SJ] and calculate the overall load vector. Show clearly how the boundary conditions may be incorporated. Assume that the global stiffness matrix for any member *m* is given by

$$[SMG]_m = \begin{pmatrix} S_{11} & \cdots & S_{16} \\ \vdots & \ddots & \vdots \\ S_{61} & \cdots & S_{66} \end{pmatrix}_{6x6}$$

   Calculation of the stiffness coefficients S_{ij} <u>is not</u> required.

   [11 marks]

   iii) Generate the global stiffness matrix [SMG] for member 1. Then, calculate the global forces at the ends of the member assuming that:

   at joint 1 the rotation is (*4.1/EI*) rad, and

   at joint 4 the rotation is (– *8.2/EI*) rad while all the other displacements are assumed zero.

   [11 marks]

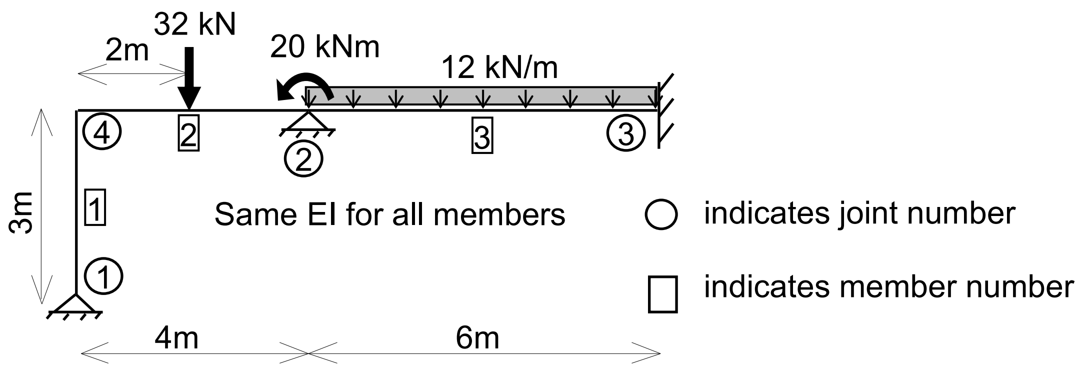

Figure Q4

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5. a) A frame is subject to the loads shown in Figure Q5. The plastic moment for each member's cross-section is given in the Figure.

Consider the sway elementary mechanism where plastic hinges are formed at Joints A, B, C and D:

i) Draw a diagram showing the collapse mechanism of the frame. Clearly indicate the key displacements on the diagram.

[5 marks]

ii) Use the method of virtual displacements to calculate the plastic collapse load.

[9 marks]

iii) Calculate the moment at the remaining critical sections then briefly comment on the results.

[11 marks]

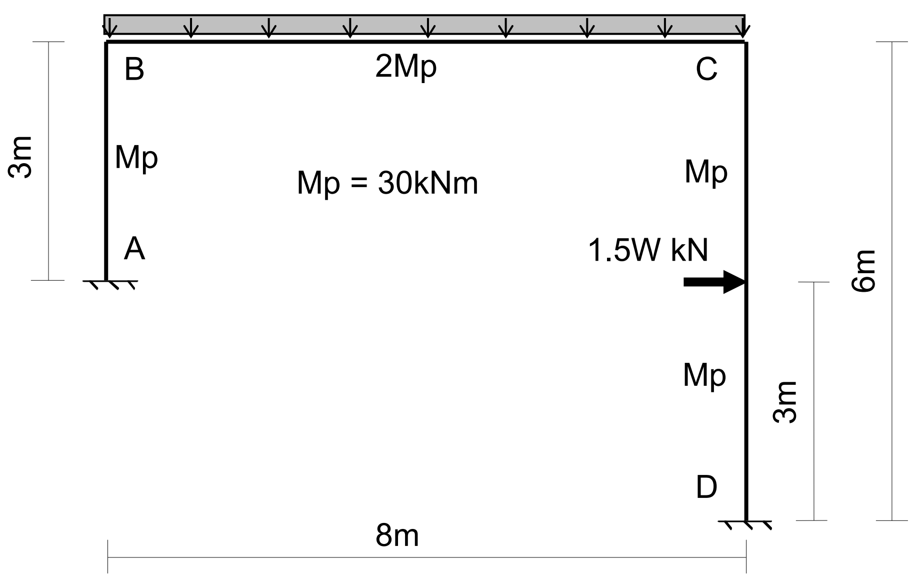

Figure Q5

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6. a) The equation of the losses $\Delta P/P$ due to friction in post tensioned beams is given by:

$$\frac{\Delta P}{P} = 1 - e^{-\mu(\theta + kx)}$$

Use the above equation to calculate the average percentage loss of prestress due to the friction component only for the beam shown in Figure Q6a. Calculate the friction losses $\Delta P/P$ at three locations of x = 0, x = 20 m and x = 40 m, then take the average. Assume the coefficient of friction $\mu = 0.2$, and the wobble coefficient per unit length of the cable $k = 0.01$/m.

[12 marks]

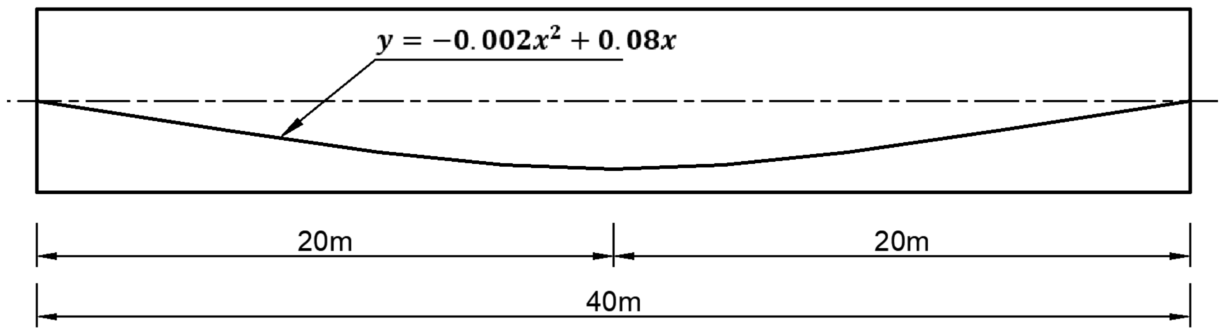
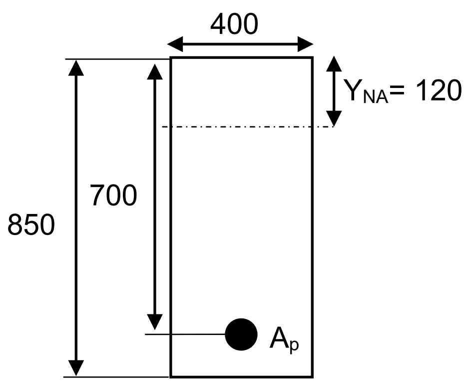


Figure Q6a

b) Draw the bending stress and strain distributions in cross-section shown in Figure Q6b and calculate the equivalent tension force in the tendon and compression force in the concrete assuming λ = 0.9 and η = 1.0. The prestress force acting on the section (after all losses) is 1200 kN. In the first trial, assume Y_{NA} is 120 mm. If the forces are not in equilibrium, should the value of Y_{NA} be increased or decreased in the second trial? What is the reason for your answer?

[13 marks]



all dimensions are in mm

Properties of concrete:
f_{ck} = 40 MPa; E_c = 35 GPa,
ε_{cu} = 0.0035

Prestressing steel:
A_p=1000 mm^2; E_p=200 GPa
f_y = 1391 N/mm^2
Losses= 20%

Figure Q6b

A El-Hamalawi, T A Dijkstra, J El-Rimawi, M W Frost, M Shaheen

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Formula Sheet for Further Structural Analysis and Geotechnical Design (CVC101)

<u>Piling:</u>

$$Qult = C_{ud} N_c Sc + \sigma_{vd}' NqSq$$

where q_{ult} = ultimate bearing capacity
B = width of foundation
σ_{vd}' = effective overburden pressure at foundation level
u = ground water pressure at foundation level
c_d = cohesion of soil below foundation
γ' = effective unit weight

$R_{cd} = R_{bk} + R_{sk} = A_b\, q_{ult} + A_s\, C_a$

where Rcd = ultimate characteristic pile resistance, at surface
A_b = area of pile base
q_{ult} = ultimate bearing capacity at base
A_s = area of surface of pile shaft
C_a = ultimate shaft friction

Piles are round or square, so **s_q = 1.2**

<u>Clay</u>:

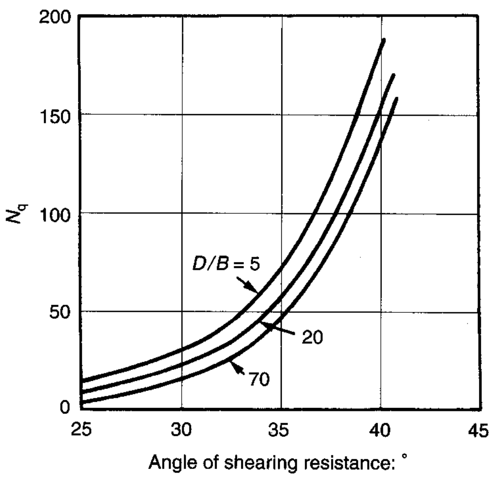
q_{ult} = $9C_b$ where C_b = design shear strength of clay at base ($C_u\omega$ for bored)
C_a = $\alpha\, C_{ave}$ where C_{ave} = average design shear strength of clay adjacent to shaft
α = adhesion factor

<u>Frictional materials</u>:

q_{ultnet} = $q'(N_q)\, s_q$
C_a = $K_s\; p'_{ave} \tan \delta$

where Ks = earth pressure coefficient
q' = effective overburden pressure at the pile base
p'_{ave} or $\overline{\sigma'}_{vd}$ = average effective overburden pressure along pile shaft
δ = angle of pile/soil friction

Pile type	δ	K_s (depending on relative density of soil)
Steel	20°	0.5 - 1.0
Concrete	0.75ϕ	1.0 – 2.0
Timber	0.67ϕ	1.5 – 4.0 (use 2.5)



Berezantsev's Nq Factors

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$R\ group_k = A_{bg}\ N_c\ C_{ud} + Asg\ C_{ud\ ave}$

$R\ group = n\ R_{cd}\ \eta$

Lateral Earth Pressure and Retaining Walls

$K_a = (1 - \sin\phi') / (1 + \sin\phi')$

$K_p = (1 + \sin\phi') / (1 - \sin\phi')$

$p_a = K_a.\ \sigma_z' - 2.c'\ \sqrt{K_a}$

$p_p = K_p.\ \sigma_z' + 2.c'\ \sqrt{K_p}$

$R_s = c_w'.\ B + V.\tan\delta'$

$Q = P/A \pm 6M/B^2L$ When Resultant is in Middle Third

Slope Stability:

Translational slide

$$F.of\ S. = \frac{c' + (\gamma.z - u)\cos^2\beta\tan\phi'}{\gamma.z\sin\beta\cos\beta}$$

Bishop's Method

$$F.of\ S. = \frac{1}{\sum W\sin\alpha}\sum\frac{[c'b + (W - u.b)\tan\phi']\sec\alpha}{1 + \frac{\tan\alpha.\tan\phi'}{F}}$$

$$W = A\frac{\gamma_K}{\gamma_\gamma} \qquad C = \frac{c'_K b}{\gamma_c} + \frac{\tan\varphi'_K}{\gamma_\varphi}\left(A\frac{\gamma_K}{\gamma_\gamma} - ub\right) \qquad D = \frac{\sec\alpha}{1 + \frac{\tan\alpha\tan\phi_K'}{\gamma_\varphi}}$$

Partial factors for the GEO ultimate limit state, **Design Approach 1**.

Combination 1

A1			**M1**		**R1**
permanent	unfavourable	1.35	γ□'	1.0	1.0
γ_G	favourable	1.0	$\gamma_{c'}$	1.0	
variable	unfavourable	1.5	γ_{cu}	1.0	
γ_Q	favourable	1.0	γ□	1.0	

Combination 2

A2			**M2**		**R1**
permanent	unfavourable	1.0	γ□'	1.25	1.0
γ_G	favourable	1.0	$\gamma_{c'}$	1.25	
variable	unfavourable	1.3	γ_{cu}	1.4	
γ_Q	favourable	0	γ□	1.0	

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Bearing Capacity

$$Qult = C_d N_c S_c + q' N_q S_q + \tfrac{1}{2}\gamma' B N_\gamma S_\gamma$$

where q_{ult} = ultimate bearing capacity
B = width of foundation
q' = effective overburden pressure at foundation level
u = ground water pressure at foundation level
c_d = design cohesion of soil below foundation
γ' = effective unit weight

Bearing Capacity Factors

ϕ	N_c	N_q	N_γ
0	5.14	1.0	0
1	5.4	1.1	0
2	5.6	1.2	0
3	5.9	1.3	0
4	6.2	1.4	0
5	6.5	1.6	0.1
6	6.8	1.7	0.1
7	7.2	1.9	0.2
8	7.5	2.1	0.2
9	7.9	2.3	0.4
10	8.4	2.5	0.5
11	8.8	2.7	0.7
12	9.3	3.0	0.8
13	9.8	3.3	1.1
14	10.4	3.6	1.3
15	11.0	3.9	1.6
16	11.6	4.3	1.9
17	12.3	4.8	2.3
18	13.1	5.3	2.8
19	13.9	5.8	3.3
20	14.8	6.4	4.0
21	15.8	7.1	4.7
22	16.9	7.8	5.5
23	18.1	8.7	6.5
24	19.3	9.6	7.6
25	20.7	10.7	9.1
26	22.3	11.9	10.5
27	23.9	13.2	12.4
28	25.8	14.7	14.5
29	27.9	16.4	17.1
30	30.1	18.4	20.0
31	32.7	20.6	23.6
32	35.5	23.2	27.7
33	38.6	26.1	32.5
34	42.2	29.4	38.4
35	46.1	33.3	45.2
36	50.6	37.8	53.3
37	55.6	42.9	63.2
38	61.4	48.9	74.9
39	67.9	56.0	89.1
40	75.3	64.2	106.0
41	83.9	73.9	126.8
42	93.7	85.4	152.0
43	105.1	99.0	182.8
44	118.4	115.3	220.8
45	133.9	134.9	267.7
46	152.1	158.5	326.3
47	173.6	187.2	399.3
48	199.3	222.3	491.6
49	229.9	265.5	608.5
50	266.9	319.1	758.0

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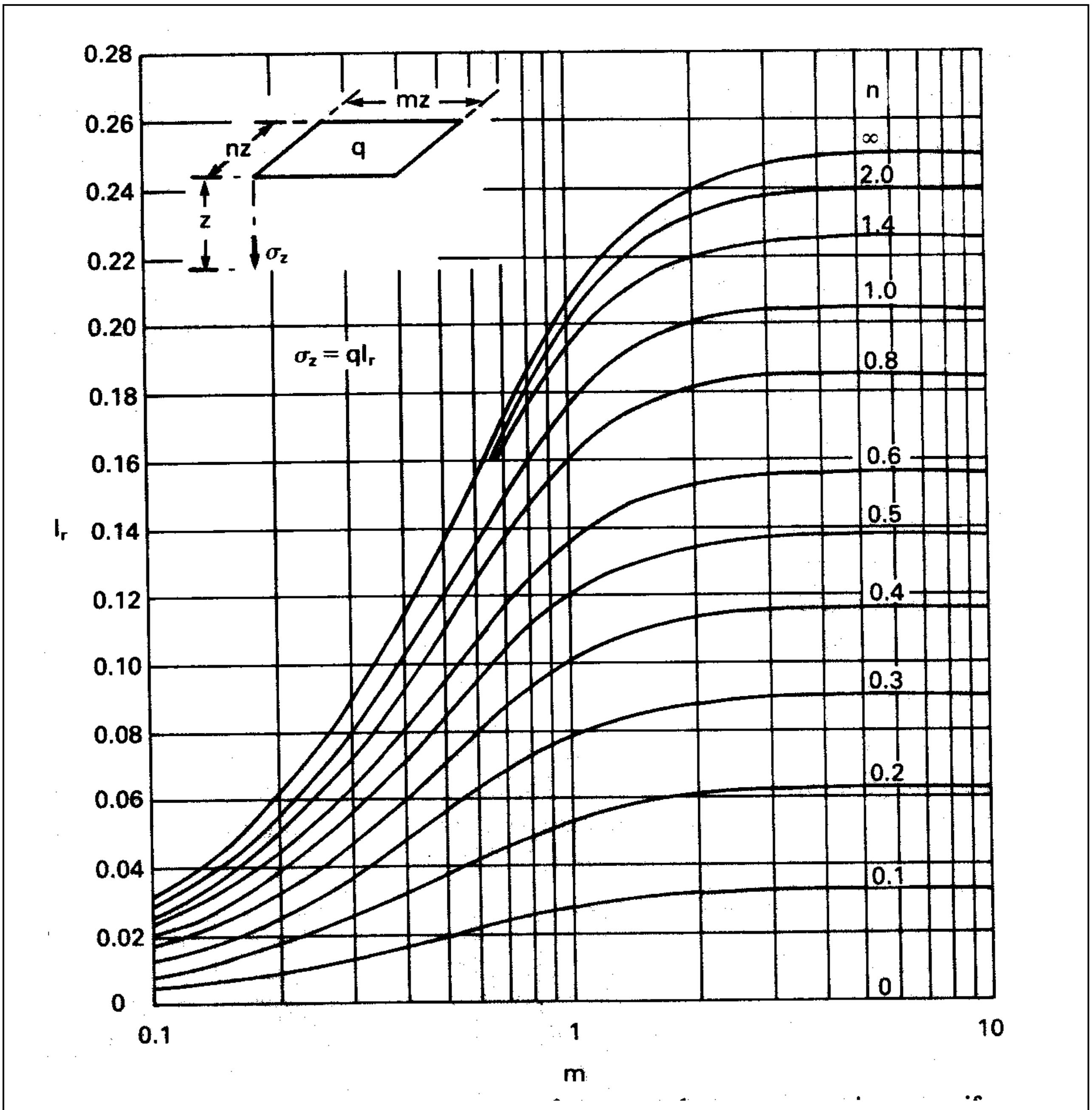
Shape factors

Shape of foundation		s_c	s_q	s_γ
strip		1.0	1.0	1.0
rectangle	Drained	$(s_q N_q - 1)/(N_q - 1)$	$1 + \frac{B'}{L'} \sin\phi$	$1 - 0.3 \frac{B'}{L'}$
	Undrained	$1 + 0.2 \frac{B'}{L'}$		
circle or square	Drained	$(s_q N_q - 1)/(N_q - 1)$	$1 + \sin\phi$	0.7
	Undrained	1.2		



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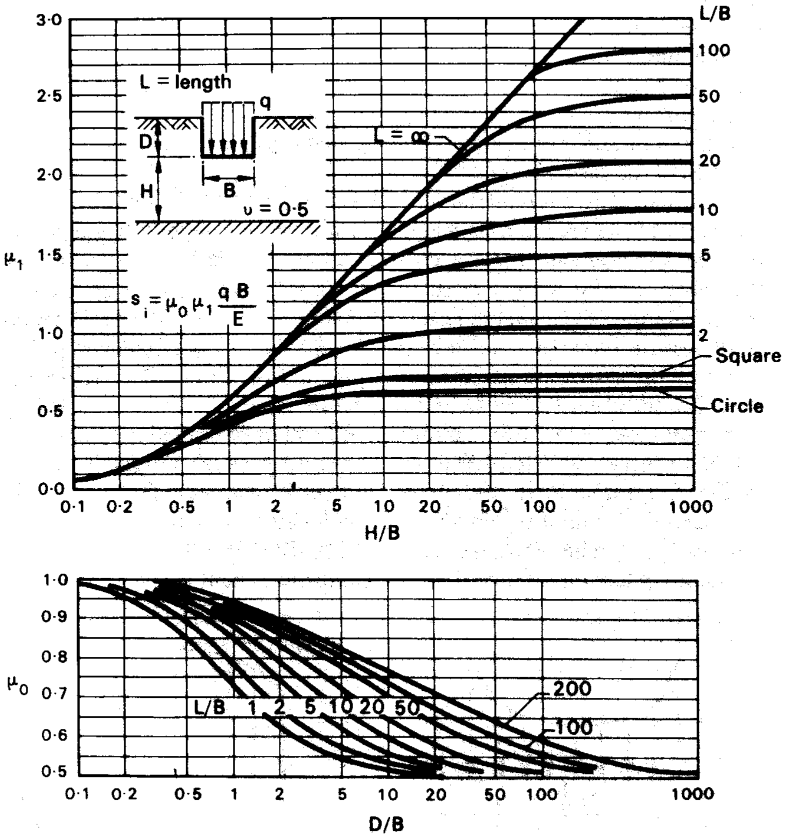

Janbu Chart

Structure	Design Approach			
	1		2	3
	Combination 1	Combination 2		
Axially loaded piles and anchors	**A1**+M1+R1	A2+(M1$^{\#}$/**M2**$^{\$}$)+**R4**	**A1**+M1+**R2**	(**A1***/**A2**†)+**M2**+**R3**
Other structures		A2+**M2**+R1		
Slopes			**A1**+M1+**R2**	**A2***†+**M2**+**R3**

$^{\#}$ for calculating resistance; $^{\$}$ for calculating unfavourable actions (e.g. down-drag)
* on structural actions; † on geotechnical actions

- In EN 1997-1, the sets of partial factors are labelled according to whether the partial factors apply to actions (A), material properties (M), or resistances (R). Where factors on actions are applied to the effect of actions rather than the actions themselves, the set is underlined (e.g. for slopes using Design Approach 2, set A1).
- Many of the partial factors given in EN 1997-1 are equal to 1,0 (and therefore can be omitted from calculations). The sets of partial factors that provide the main source of safety (i.e. have values other than 1,0) in a particular combination are shown in **bold**. For example, when using Design Approach 1/combination 1, safety is introduced primarily through factors on actions **A1**. In DA1/combination 2 for slopes, safety is introduced primarily through material factors **M2**.

Extracts from EC7 Design Cases

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Partial factors on actions for different limit states

Duration of action	Effect of action	Symbol γ_F	Limit state/partial factor set				
			EQU	STR/GEO		UPL	HYD
				A1	A2		
Permanent	Unfavourable	$\gamma_{G;dst}$	1,1	1,35	1,0	1,0	1,35
	Favourable	$\gamma_{G;stb}$	0,9	1,0	1,0	0,9	0,9
Variable	Unfavourable	$\gamma_{Q;dst}$	1,5	1,5	1,3	1,5	1,5

Unfavourable actions (with the subscript "dst" above) are those which destabilize the structure and favourable actions (subscript "stb") are those which stabilize the structure. Variable, favourable actions are omitted from the table above because they are deliberately ignored in EN 1997-1 (i.e. $\gamma_{Q;stb} = 0$).

Example (using limit state STR/GEO partial factor set A1)
If the representative vertical load (F_{rep}) on a footing is 100 kN, then the design vertical load (F_d) would be 100 x 1.35 = 135 kN.

Extracts from EC7 Action Factors

Soil parameter	Symbol γ_M	Limit state/partial factor set				
		EQU	STR/GEO		UPL	HYD
			M1	M2		
Angle of shearing resistance	$\gamma_{\phi'}$	1,25*	1,0*	1,25*	1,25*	–
Effective cohesion	$\gamma_{c'}$	1,25	1,0	1,25	1,25	–
Undrained shear strength	γ_{cu}	1,4	1,0	1,4	1,4	–
Unconfined strength	γ_{qu}	1,4	1,0	1,4	1,4	–
Weight density	γ_{γ}	1,0	1,0	1,0	–	–
Tensile pile resistance	$\gamma_{s;t}$	–	–	–	1,4	–
Anchorage	γ_R	–	–	–	1,4	–

*Applied to tan ϕ' not ϕ'

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Extracts from EC7 Material Factors

In equation (7.4), $R_{c;k}$ is the characteristic value of the compressive resistance of the pile and γ_t is a partial factor on that resistance.

In equation (7.5), $R_{b;k}$ is the characteristic base resistance of the pile, $R_{s;k}$ is its characteristic shaft resistance, γ_b is a partial factor on the base resistance and γ_s is a partial factor on the shaft resistance.

Values of γ from the National Annex to BS EN 1997-1 are given below. Please note that these values differ significantly from those given in EN 1997-1 Annex A, and the figures are provisional at the time of writing (July 2007). With these factors, equation 7.4 always gives design resistances equal to or lower than equation 7.5.

Partial factors for piles in compression

Resis-tance	Symbol	Partial factor set for different pile types				
		R1	R4			
			Without load tests*		With load tests*	
		All types	Bored & CFA	Driven	Bored & CFA	Driven
Base	γ_b	1,0	2,0	1,7	1,7	1,5
Shaft	γ_s		1,6	1,5	1,4	1,3
Total	γ_t		2,0	1,7	1,7	1,5

* The lower values of γ_b, γ_s, and γ_t in R4 may be adopted if serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1,5 times the representative load for which they are designed, or if settlement at the serviceability limit state is of no concern.

Extracts from EC7 Pile Resistance Factors

(Note. For design Approach 1, Resistance factors for R1 and R2 are normally = 1)

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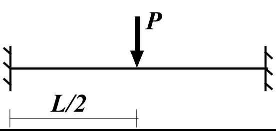
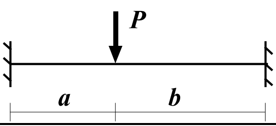
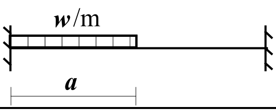
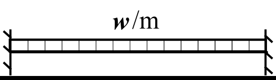
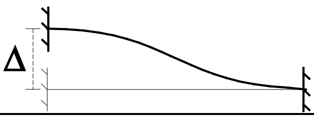
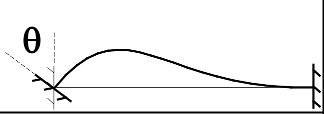
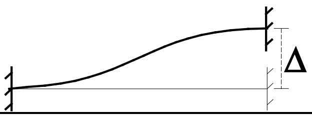
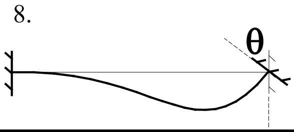
Fixed End Forces

a. Due to in-span loads

	FEL_1	FEL_2	FEL_3	FEL_4
1. 	$\frac{P}{2}$	$\frac{PL}{8}$	$\frac{P}{2}$	$-\frac{PL}{8}$
2. 	$\frac{Pb^2}{L^3}(3a+b)$	$\frac{Pab^2}{L^2}$	$\frac{Pa^2}{L^3}(3b+a)$	$-\frac{Pba^2}{L^2}$
3. 	$\frac{wa}{2L^3}(2L^3-2a^2l+a^3)$	$\frac{wa^2}{12L^2}(6L^2-8al+3a^2)$	$\frac{wa^3}{2L^3}(2L-a)$	$-\frac{wa^3}{12L^2}(4L-3a)$
4. 	$\frac{wL}{2}$	$\frac{wL^2}{12}$	$\frac{wL}{2}$	$-\frac{wL^2}{12}$

b. Due to joint displacement

5. 	$\frac{12EI}{L^3}\Delta$	$\frac{6EI}{L^2}\Delta$	$-\frac{12EI}{L^3}\Delta$	$\frac{6EI}{L^2}\Delta$
6. 	$\frac{6EI}{L^2}\theta$	$\frac{4EI}{L}\theta$	$-\frac{6EI}{L^2}\theta$	$\frac{2EI}{L}\theta$
7. 	$-\frac{12EI}{L^3}\Delta$	$-\frac{6EI}{L^2}\Delta$	$\frac{12EI}{L^3}\Delta$	$-\frac{6EI}{L^2}\Delta$
8. 	$\frac{6EI}{L^2}\theta$	$\frac{2EI}{L}\theta$	$-\frac{6EI}{L^2}\theta$	$\frac{4EI}{L}\theta$
9. 	$FEL_1 = -\frac{EA}{L}\Delta$		$FEL_2 = \frac{EA}{L}\Delta$	