

Subframe analysis

Floor beam of two to five spans with the columns above and below · EN 1992-1-1 §5.1.3, §5.4 · UK NA / PN-EN

Reference	NAE-RC-025 · version 1.0
Standards	BS EN 1992-1-1:2004+A1:2014 / PN-EN 1992-1-1:2008, 5.1.3 and 5.4 (linear elastic analysis) · BS EN 1990 / PN-EN 1990 (6.10)
National Annex	Selectable load arrangement: UK NA to BS EN 1992-1-1, NA.2 5.1.3(1)P (all spans and alternate spans loaded) or the recommended arrangements of the note to 5.1.3(1)P (alternate spans and any two adjacent spans loaded), used for PN-EN: the National Annex of PN-EN 1992-1-1:2008 makes no choice for 5.1.3(1)P and keeps the recommended provisions (NA.9); confirm this against the annex edition and amendments adopted for the project (e.g. NA:2018-11)
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What it does

Linear elastic analysis of a floor beam of two to five spans built into the columns above and below every support, the far ends of the columns fixed and no sway: the subframe of the usual building design practice (IStructE Manual, BS 8110-1 3.2.1.2). The sheet gives the envelope of the beam span and support moments, the hogging at the end supports from the frame action, the moments in the columns at every node, the largest shears and the reactions, with the frame and the moment envelope drawn. Use the beam moments and shears in NAE-RC-004 Beam bending and shear and the column moments in NAE-RC-008 Column design as one input to the column design, not the column design itself.

How to use

1. Open the Calculation sheet and choose the load arrangement and the number of spans.
2. Enter the span lengths, the beam and column sections, the storey heights and whether there are columns above and below. Spans beyond the number chosen, and the column data when there are no columns, are ignored and may be left blank.
3. Enter one permanent and one variable UDL, common to every span, and the partial factors. Read the Results block, the moments at every node and the drawing.

Scope and limitations

No sway: the frame is braced and the horizontal loads are carried elsewhere. Prismatic members with gross uncracked rectangular sections and one modulus of elasticity (5.4(2)); the same column section and storey heights at every node; the far ends of the columns fixed; a knife-edge support where there is no column below; axial shortening ignored. Uniform loads over every span, one γ_G for every span. Load arrangements: UK NA option of all spans and alternate spans loaded (cases 1 – 3), or EN / PN-EN alternate spans and any two adjacent spans (cases 2 – 3 and 4 onwards); case 1, all spans loaded, is kept in the EN set too, which can only enlarge the envelopes of this model. The slope-deflection equations are solved exactly. Moments are at the centre lines of the members; the beam may be designed for the moment at the face of the column, not less than 0.65 of the fixed-end moment (5.3.2.2(3)); face moments and that minimum are not calculated here. Moment redistribution (5.5) is not applied. The column moments are the largest magnitudes $|M|$ over the cases, first order and from this floor only; in this model the fixed far end carries half the moment at the node, in double curvature, but that is not the moment diagram of the real column: its end moments come from the subframes of the floors at both of its ends. The column design (NAE-RC-008 or a frame analysis) must establish the end moments of each storey, the axial load (NAE-RC-019), the imperfections (5.2), the slenderness and second-order effects (5.8) and the minimum eccentricity (6.1(4)). A negative reaction (uplift) is flagged; it is not an EQU verification. A flanged beam may be entered with an equivalent rectangular depth: only the ratio of the beam and column stiffnesses matters. The second-generation EN 1992-1-1:2023 is not yet implemented.

Further reading and independent cross-check

- IStructE, Manual for the design of concrete building structures to Eurocode 2: subframe analysis of beams and columns.
 - The Concrete Centre, How to design concrete structures using Eurocode 2: Beams.
 - UK NA to BS EN 1992-1-1, NA.2 5.1.3(1)P: load arrangements.
 - NAE-RC-018 Continuous beam gives the same beam on knife-edge supports: with no columns this sheet reproduces it.
- NA Engineering is not affiliated with The Concrete Centre or CIRIA; their publications are referenced for information only.

About NA Engineering

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Revision history

v1.0	29 Oct 2026 · First issue
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Project
Element
Date

Job no.
Made by
Checked

Inputs · frame

Shaded cells only

Load arrangement (5.1.3(1)P)	UK NA	(all and alternate spans)	
Number of spans	n	3	
Span 1, centres of columns	L_1	6000	mm
Span 2, centres of columns	L_2	5000	mm
Span 3, centres of columns (used when $n \geq 3$)	L_3	6000	mm
Span 4, centres of columns (used when $n \geq 4$)	L_4	5000	mm
Span 5, centres of columns (used when $n \geq 5$)	L_5	6000	mm
Beam width	b	300	mm
Beam overall depth (gross section)	h	600	mm
Columns above every node?		Yes	
Columns below every node?		Yes	
Column width (at right angles to the frame)	b_c	400	mm
Column depth (in the plane of the frame)	h_c	400	mm
Storey height above, floor to floor (with columns above)	H_{up}	3500	mm
Storey height below, floor to floor (with columns below)	H_{dn}	3500	mm

Inputs · loads

Characteristic values

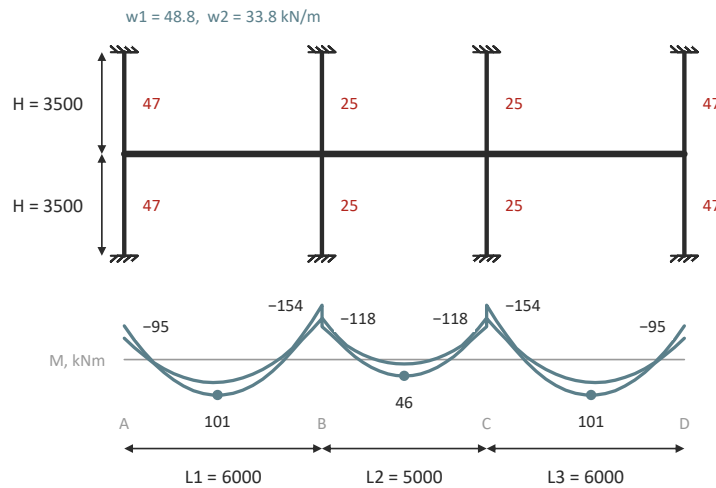
	UDL, permanent (include the self weight)	g_k	25.00	kN/m
	UDL, variable	q_k	10.00	kN/m
EN 1990 A1.2(B)	Partial factor, permanent	γ_G	1.35	
EN 1990 A1.2(B)	Partial factor, variable	γ_Q	1.50	
Inputs valid and consistent				OK

Results

Largest sagging moment (span)	$M_{Ed,span}$	100.7	kNm
Largest hogging moment (interior nodes)	$M_{Ed,sup}$	153.5	kNm
Largest hogging moment (end nodes, frame action)	$M_{Ed,end}$	94.9	kNm
Largest column M (edge columns)	$M_{c,edge}$	47.5	kNm
Largest column M (interior columns)	$M_{c,int}$	25.4	kNm
Largest shear force (at a node)	$V_{Ed,max}$	156.5	kN
Largest reaction	R_{max}	278.4	kN
Smallest reaction	R_{min}	92.0	kN

OK

Your subframe · largest column |M| at the nodes over the cases (kNm, red) · beam moment envelope below (kNm, sagging down)



Calculation · loads and stiffness

(6.10)	Design load, loaded span $\gamma_G g_k + \gamma_Q q_k$	w_1	48.75	kN/m
5.1.3	Design load, unloaded span $\gamma_G g_k$	w_2	33.75	kN/m
5.4(2)	Beam, gross section $I_b = b h^3 / 12$	I_b	5400.0	10 ⁶ mm ⁴
5.4(2)	Column, gross section $I_c = b_c h_c^3 / 12$ (0: no columns)	I_c	2133.3	10 ⁶ mm ⁴
	Column above, far end fixed $4 I_c / H_{up}$ (0: none)	k_{up}	2438.1	10 ⁶ mm ⁴ /m
	Column below, far end fixed $4 I_c / H_{dn}$ (0: none)	k_{dn}	2438.1	10 ⁶ mm ⁴ /m
	Columns at a node $k_c = k_{up} + k_{dn}$	k_c	4876.2	10 ⁶ mm ⁴ /m
	End node: share in the columns $k_c / (k_c + 4 I_b / L_1)$	δ_c	0.58	
5.1.3(1)P	Number of load cases analysed	n_c	3	

Load cases: 1 every span carries w_1 ; 2 odd spans w_1 , even spans w_2 ; 3 even spans w_1 , odd spans w_2 ; with the EN / PN-EN arrangement also 4 ... $n + 2$: spans k and $k + 1$ carry w_1 , the others w_2 ($k = 1 \dots n - 1$). Slope-deflection with the node rotations ϑ (E common): node j $2 k_j \vartheta_{j-1} + (4 k_j + 4 k_{j+1} + k_c) \vartheta_j + 2 k_{j+1} \vartheta_{j+1} = w_{j+1} L_{j+1}^2 / 12 - w_j L_j^2 / 12$, $k = I_b / L$. Beam end moments: left $-w L^2 / 12 + 2 k (2 \vartheta a + \vartheta b)$, right $-w L^2 / 12 - 2 k (\vartheta a + 2 \vartheta b)$; column moments at the node $k_{up} \vartheta$ and $k_{dn} \vartheta$, half that at the fixed far end. The envelope is the worst of the cases at every point.

Beam moments are signed: + sagging (tension at the bottom), - hogging. At a node the beam moments either side differ by the moment taken by the columns.

Calculation · beam moments at the nodes (column centre lines)

5.4	Node A: beam moment right of the node, most negative	$M_{A,r}$	-94.9	kNm
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5.4	Node B: beam moment left of the node, most negative	$M_{B,l}$	-153.5	kNm
5.4	Node B: beam moment right of the node, most negative	$M_{B,r}$	-117.5	kNm
5.4	Node C: beam moment left of the node, most negative	$M_{C,l}$	-117.5	kNm
5.4	Node C: beam moment right of the node, most negative	$M_{C,r}$	-153.5	kNm
5.4	Node D: beam moment left of the node, most negative	$M_{D,l}$	-94.9	kNm
5.4	Node D: beam moment right of the node, most negative	$M_{D,r}$	—	kNm
5.4	Node E: beam moment left of the node, most negative	$M_{E,l}$	—	kNm
5.4	Node E: beam moment right of the node, most negative	$M_{E,r}$	—	kNm
5.4	Node F: beam moment left of the node, most negative	$M_{F,l}$	—	kNm

Calculation · spans, shears and reactions

5.4	Largest moment in span 1, worst of the cases	M_1	100.7	kNm	at 2.83 m
5.4	Largest moment in span 2, worst of the cases	M_2	46.3	kNm	at 2.50 m
5.4	Largest moment in span 3, worst of the cases	M_3	100.7	kNm	at 3.17 m
5.4	Largest moment in span 4, worst of the cases	M_4	—	kNm	
5.4	Largest moment in span 5, worst of the cases	M_5	—	kNm	
	Largest shear at node A	V_A	138.1	kN	
	Largest shear at node B	V_B	156.5	kN	
	Largest shear at node C	V_C	156.5	kN	
	Largest shear at node D	V_D	138.1	kN	
	Largest shear at node E	V_E	—	kN	
	Largest shear at node F	V_F	—	kN	
	Reaction at node A (max / min)	R_A	138.1 / 92.0	kN	
	Reaction at node B (max / min)	R_B	278.4 / 232.4	kN	
	Reaction at node C (max / min)	R_C	278.4 / 232.4	kN	
	Reaction at node D (max / min)	R_D	138.1 / 92.0	kN	
	Reaction at node E (max / min)	R_E	—	kN	
	Reaction at node F (max / min)	R_F	—	kN	

Calculation · column moments at the nodes (largest |M| of the cases)

$|M|$ is the largest magnitude over the cases. Sense: the columns at a node balance the step in the beam moment, $M_l - M_r = M_{up} + M_{dn}$ (beam moments + sagging; column end moments clockwise +), so the sign follows the rotation of the node and may change from case to case at interior nodes.

5.4	Node A: column above, largest M at the node	$M_{A,up}$	47.5	kNm
5.4	Node A: column below, largest M at the node	$M_{A,dn}$	47.5	kNm
5.4	Node B: column above, largest M at the node	$M_{B,up}$	25.4	kNm
5.4	Node B: column below, largest M at the node	$M_{B,dn}$	25.4	kNm
5.4	Node C: column above, largest M at the node	$M_{C,up}$	25.4	kNm
5.4	Node C: column below, largest M at the node	$M_{C,dn}$	25.4	kNm
5.4	Node D: column above, largest M at the node	$M_{D,up}$	47.5	kNm
5.4	Node D: column below, largest M at the node	$M_{D,dn}$	47.5	kNm
5.4	Node E: column above, largest M at the node	$M_{E,up}$	—	kNm
5.4	Node E: column below, largest M at the node	$M_{E,dn}$	—	kNm
5.4	Node F: column above, largest M at the node	$M_{F,up}$	—	kNm
5.4	Node F: column below, largest M at the node	$M_{F,dn}$	—	kNm

Calculation · summary

	Largest sagging moment (0: none)	M_{span}	100.72	kNm
	Largest hogging at an interior node (0: none)	M_{sup}	153.54	kNm
	Largest hogging at an end node (0: none)	M_{end}	94.94	kNm
	Largest column M at an end node, above or below	$M_{c,edge}$	47.47	kNm
	Largest column M at an interior node, above or below	$M_{c,int}$	25.44	kNm
	Largest shear	V_{max}	156.54	kN
	Largest reaction	R_{max}	278.42	kN
	Smallest reaction	R_{min}	91.96	kN

Elastic moments at the centre lines, no redistribution. 5.3.2.2(3): where the beam is monolithic with the column, the critical support moment may be taken at the column face, not less than 0.65 of the fixed-end moment (neither is calculated here); the moment and reaction passed to the column are the elastic values given here. Shears are at the column centre lines; 6.2.1(8) allows the check at d from the face for a mainly uniform load.

Columns: the moments are first order, from this floor only; in this model the fixed far end carries half the moment at the node, in double curvature. They are one input to the column design, not the column design: the end moments of each storey also depend on the floor at its other end; check slenderness and second order effects (5.8), the imperfections (5.2) and the minimum eccentricity $e_0 = \max(h / 30; 20 \text{ mm})$ (6.1(4)) in NAE-RC-008 or a frame analysis.

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