

FEAKalc 3D

Shear Wall Design Theory Manual to AS 3600 2018 | Revision 0

Document Revision

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This manual describes the theory and application of the FEAkalc 3D shear wall design routines.

Read the strength results together with their action coverage, boundary elements screening and detailing requirements.

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1. Introduction

1.1. Purpose and coverage

FEAKalc 3D designs labelled reinforced concrete wall piers and spandrels using actions recovered from the three dimensional finite element model.

This manual explains the theory behind that process, the equations used to calculate reinforcement and utilisation, the meaning of the reported results, and the assessments that remain with the structural designer. It is intended to be read alongside the model, the selected ultimate load combinations, and the member calculation memos.

The pier procedure selects a common two face longitudinal reinforcement arrangement for axial force and bending about both section axes, while checking in plane shear and selecting horizontal reinforcement. It also screens the two wall edges for boundary restraint using gross elastic stresses. The spandrel procedure uses a coupling beam strut and tie model for shear ties and nodal stresses, with a separate longitudinal tie calculation based on the analysis end moments.

The methods described are those implemented in the project source as reviewed on 28 October 2026. Their design basis is AS 3600:2018 incorporating Amendments 1 and 2.

The 2026 public comment draft is not applied. Code references identify the basis of particular checks; the equations and qualifications in this manual describe the actual calculation adopted by FEAKalc 3D.

1.2. Relationship between analysis and design

The finite element solver establishes the load path through the wall system. It determines shell displacements and element nodal actions for each analysis case. A separate recovery process assembles those actions into member end resultants, which the design routines then use.

Assigning a pier or spandrel label is a way of defining the member over which actions are integrated; it does not introduce a different structural element or automatically modify the stiffness of the analysis model.

The design engine does not reconstruct the applied axial force from dead and live loads or infer an equivalent cantilever moment from building height. It uses the supplied factored member actions. Section minimum eccentricities, optional out of plane magnification, and nominated shear amplification are applied subsequently and are reported separately from the original actions.

The distinction matters where a model contains coupled walls, irregular loading or several selected combinations. The largest axial force, largest moment and largest shear may occur in different combinations.

Each selected pier section tuple is checked as a row, and a common reinforcement arrangement is sought across the available rows. Spandrel shear and longitudinal reinforcement use independent envelopes.

1.3. Structural idealisation

- A pier is the vertical wall member defined by its assigned source shells.
- A spandrel is the horizontal wall member, commonly a coupling beam between adjacent piers.
- A label covers its complete assigned extent.
- Automatic analysis meshing and manual subdivisions do not create additional design ends. If storey by storey checks are required, the designer must define suitable separate labels and assess the force transfer between them.
- Refer to figures below for demonstration of piers and spandrels in FEAKalc, and presentation of design results on the screen.

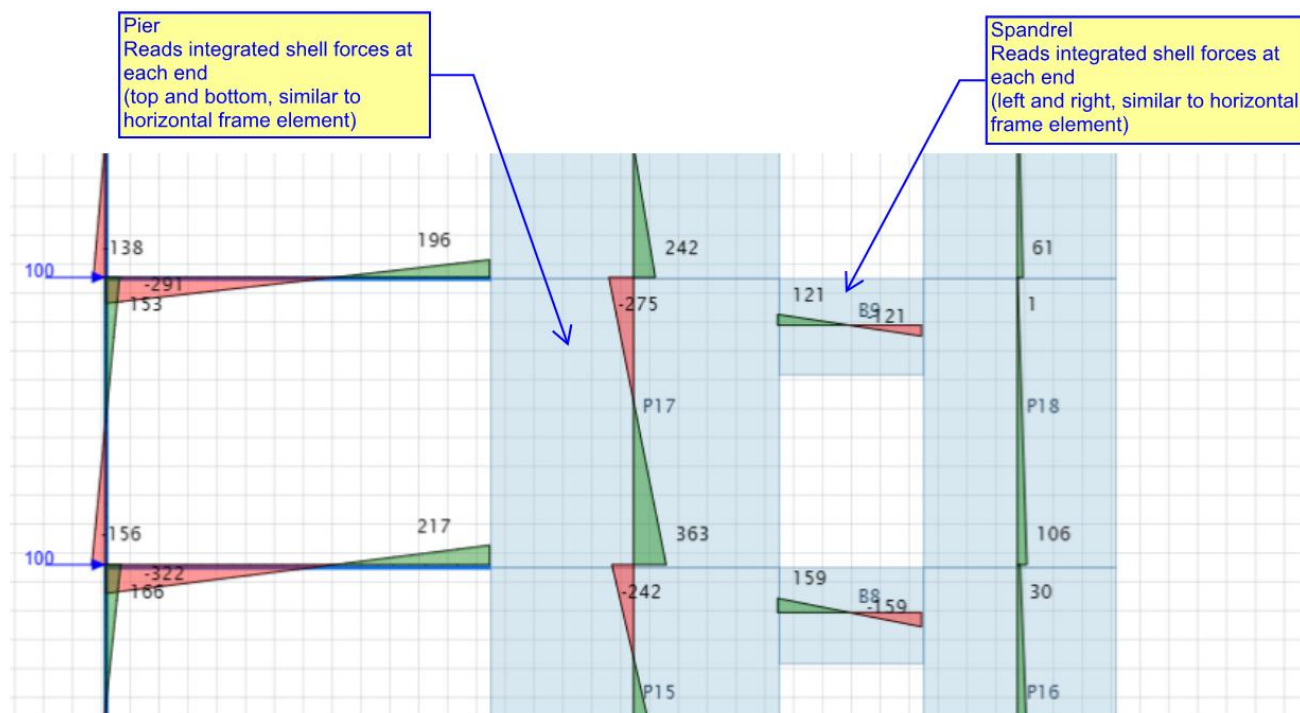


Figure 1.1 – Piers and Spandrels in FEAkalc

Current reinforcement design requires a straight, planar, complete rectangular member of uniform thickness and concrete strength. The force recovery system has broader reporting capabilities, including some nonplanar pier sections, but those capabilities do not extend the rectangular design engine to L sections, T sections, cores, flanged walls, tapered members or members containing openings.

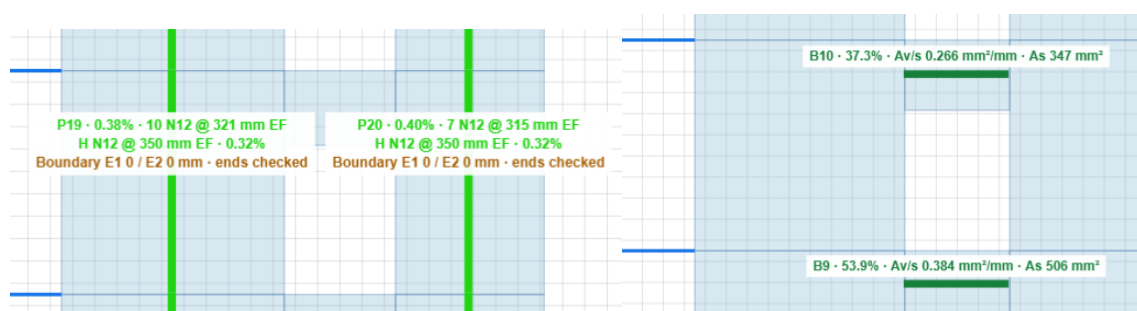


Figure 1.2 – Piers and Spandrel Design in FEAkalc

1.4. Strength design and acceptance

The general strength comparison is between a factored demand and the corresponding design resistance. Reduction factors depend on the failure mechanism. The pier section interaction curves use the inherited concrete section engine and its point dependent reduction factors. Pier shear normally uses 0.70, with a conditional 0.75 option. Spandrel steel ties **use 0.85**, while unconfined nodal stress checks use 0.65 together with the nodal efficiency factors.

$$U = \text{design demand} / \text{design resistance}$$

A pier passes the implemented section and shear checks when the largest applicable utilisation does not exceed 1.00 and the required action coverage is complete. Its boundary screening is a separate result. A spandrel passes its reported nodal checks when the maximum nodal utilisation does not exceed 100 percent; its reinforcement areas are calculated demands rather than a verified detailed bar arrangement.

An acceptable numerical result therefore has a defined scope. It does not by itself establish anchorage, lap, confinement fitment or serviceability compliance. Those requirements must be completed in the design and detailing of the structural system.

1.5. Organisation of this manual

- Chapter 2 describes member geometry, local axes and force recovery.
- Chapter 3 establishes combination processing and spectrum treatment.
- Chapters 4 to 6 describe pier section design, shear and boundary zones.
- Chapter 7 develops the spandrel strut and tie method.
- Chapters 8 and 9 explain result interpretation and the complete design workflow.
- Chapter 10 records assumptions and coverage.
- The appendices provide hand calculation examples, symbols and a traceable reference map.

2. Member Geometry and Finite Element Actions

2.1. Source shells and member labels

Pier and spandrel assignments are stored on the source wall objects. One source object may carry both types of label. Several objects can form one labelled member where they connect along edges. Partial common edges can provide connectivity; a point contact or a gap does not establish a continuous member for this purpose.

The design member is assembled from all source shells with the same label. Its dimensions and properties must therefore be checked over the whole label. Hiding part of a wall in a view does not remove its contribution from the member total. A complete labelled member must not inadvertently combine disconnected walls or coupling beams separated by a pier.

The geometry checks compare the area of the projected source polygons with their enclosing rectangle and check planarity. This rejects openings and taper for reinforcement design. A rectangular bounding box alone is not sufficient evidence that a wall with missing material can use the rectangular section calculation.

2.2. Pier dimensions and section axes

For a pier, local axis 1 is vertical and points in global positive Z. Local axis 2 lies along the deterministic horizontal member direction, oriented toward positive global X, or positive Y for a wall aligned with global Y. Local axis 3 is the cross product of local axes 1 and 2. The horizontal design section has length L along local 2 and thickness t along local 3.

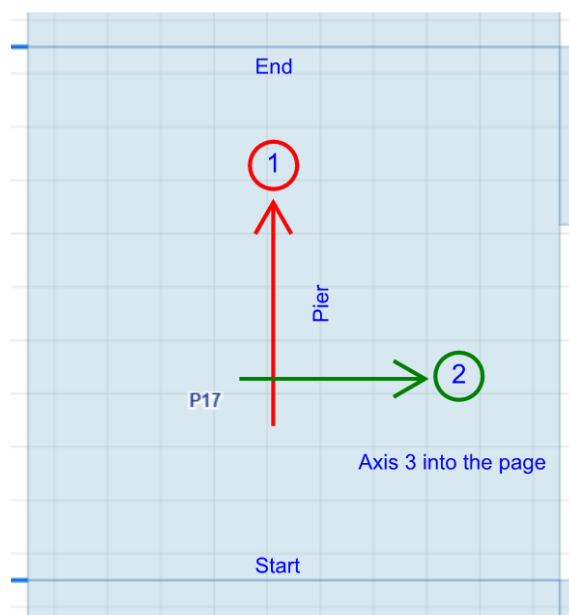


Figure 2.1 – Local Axes of Piers

The program obtains L (length) from the horizontal extent of the complete label, t from its common wall thickness, and the labelled height from its vertical extent. Model geometry is converted from metres to millimetres before it enters the design engine. The pier section is treated as a rectangle with two equal rows of vertical reinforcement near its two thickness faces.

$$A = Lt \quad I_{22} = Lt^3/12 \quad I_{33} = tL^3/12$$

M22 bends across the wall thickness and is the out of plane action. M33 bends along the wall length and is the in plane action. V2 is the integrated in plane shear used for pier design. These definitions are tied to the displayed member axes and must be checked when comparing results with another analysis program.

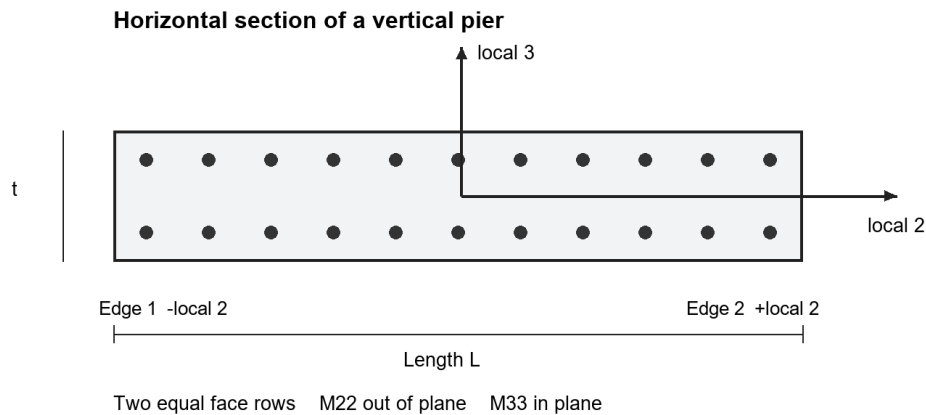


Figure 2.2- Pier section axes and reinforcement idealisation

2.3. Spandrel dimensions and axes

For a spandrel, local axis 1 follows the horizontal member direction, local axis 2 points vertically upward, and local axis 3 is their cross product. The design uses local V2 and M3. Its full projected horizontal extent is taken as the clear span l , its vertical extent as the overall depth D , and the common source thickness as t .

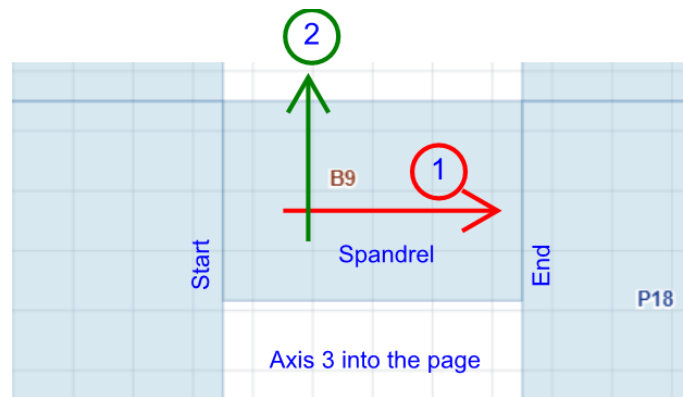


Figure 2.3 – Spandrel Local Axes

The label boundaries must consequently represent the intended coupling beam faces. Including substantial pier regions in the spandrel label changes the span and the recovered end actions. The end node width is an additional strut and tie input; it does not correct an incorrectly defined geometric clear span.

The reinforcement centre c is measured from the top or bottom concrete face to the corresponding longitudinal tie centre. The current model uses equal top and bottom centres, giving a truss depth $z = D - 2c$. Node widths and reinforcement centres must be consistent with a physical reinforcement and anchorage arrangement.

2.4. Recovery of element nodal actions

Wall design actions are recovered from element nodal forces and couples, rather than from smoothed contour stresses or element peak magnitudes. At element level the recovery uses the assembled symmetric stiffness and element displacement, with equivalent shell surface load actions subtracted for static cases.

$$f_e = K_e u_e - f_{e,load}$$

The recovered element vector contains the six global action components at each analysis node. For each member end, the recovery routine identifies aligned end edges at the minimum or maximum projection along local axis 1. The reference centroid is weighted by edge length times shell thickness.

$$\bar{r} = \Sigma(A_j r_j) / \Sigma A_j \quad A_j = \text{edge length}_j \times \text{thickness}_j$$

Forces are summed, and moments include both the nodal couples and the lever arm of every force about the section reference centroid. Omitting the lever arm contribution would lose the axial force couple that produces wall bending.

$$F = \Sigma F_i \quad M = \Sigma [M_i + (r_i - \bar{r}) \times F_i]$$

The global sums are resolved onto the member axes. Internal analysis mesh divisions do not create new reporting ends or additional section resultants. Recovery requires complete contributing mesh actions. Missing elements, missing source objects, duplicate elements or incompatible combination meshes produce errors rather than a silent partial force sum.

2.5. Sign convention

FEAKalc reports positive P as tension and negative P as compression. The pier section calculation uses compression positive, so its applied axial force is $N^* = -P$. Moments follow the right hand rule about the displayed member axes. The recovery also applies the solver sign conversion and reverses the lower or left end to express both ends using a positive face section convention.

$$N^* = -P$$

The signed values are retained during static combination processing and during separation of spectrum shear contributions. Taking absolute values of individual case actions before combination would destroy cancellation and reversal information. Magnitudes are taken only at the design envelope stage specified in the relevant chapter.

The model diagram selectors use a frame display convention that interchanges the names associated with the two integrated shear components. Design input follows the raw integrated member V2, not an assumption based solely on the displayed diagram selector. Read the force table, axes and memo when checking a shear result.

2.6. Station coverage and equilibrium

The current wall member design uses two stations: Bottom and Top for piers, Left and Right for spandrels. The straight line joining the end resultants in a diagram is a display representation; it does not provide additional calculated stations. Local concentrations around openings, changes in thickness, transfer levels and interior force peaks require separate evaluation.

For an unloaded interval of length h along local axis 1, a consistent set of signed resultants satisfies the expected end equilibrium relationships. With the adopted axes, examples are $M_{2,\text{lower}} = M_{2,\text{upper}} - hV_3$ and $M_{3,\text{lower}} = M_{3,\text{upper}} + hV_2$. Distributed loads and changing axial force require their corresponding equilibrium terms.

These relationships are useful model checks, but they do not justify replacing the recovered end moments by a shear times height expression in the pier section design. The spandrel shear model introduces an equivalent equal moment idealisation for a different, explicitly defined purpose.

3. Design Actions and Combination Envelopes

3.1. Selection of ultimate combinations

The designer selects the combinations to be covered by a design run. Each selected combination is read independently. The program does not generate unselected combinations, change their factors, or automatically create every possible sign permutation of spectrum component peaks. The engineering adequacy of the selected ULS set must therefore be established before design.

For a static linear combination, the signed member action vector is the factored sum of its case vectors. Each end retains the axial force, both bending moments and shear associated with that combination. The pier section calculation evaluates these tuples separately rather than assembling a fictitious tuple from unrelated maxima.

$$R_{\text{combo}} = \sum \lambda_k R_k$$

The same principle applies to gravity and other static contributions within a combination containing a spectrum case. Their original factors remain unchanged by the shear overstrength calculation.

3.2. Modal integration before spectrum combination

Response spectrum actions require special treatment because modal peak values do not describe one simultaneous response state. For each signed modal response, the element actions are first integrated into the member force and moment resultants. The member modal resultants are then combined using the spectrum case modal and directional rules and scaling.

$$R_{\text{member,mode}} = \text{integrate signed element actions for that mode}$$

$$R_{\text{member,spectrum}} = \text{spectrum combination of member modal resultants}$$

Summing absolute element peaks would generally exaggerate the member resultant and would not respect section equilibrium. Integrating the signed modal field first preserves the relationship between the distributed element actions and the resultant being combined.

3.3. Spectrum inputs to pier section design

The current pier workflow accepts the reported P, M22 and M33 component peaks of each selected combination as its section design inputs. Their reported signs and values are passed to the section routine. No time history is reconstructed, and no independent sign combinations are added automatically.

A spectrum row is therefore a component peak envelope input. It must not be described as a demonstrated time concurrent N M V state. The same qualification applies when the row is used for gross stress boundary screening. The memo identifies such rows so that the designer can assess whether the adopted combination convention is appropriate for the structural system.

Static rows retain their normal combination interpretation.

A design run may include both static and spectrum rows and uses one common reinforcement arrangement across the available section rows. There is no requirement to run a preliminary static design before a spectrum envelope design.

Refer to next figure for clarity of spectrum and static rows

Selected combination section actions — including spectrum peaks								
Analysis P is positive in tension; section compression N* = -P. M22 = integrated M2 (out of plane); M33 = integrated M3 (in plane). Spectrum rows are checked as the selected combination component-peak envelope, not as a time-concurrent response history. Static rows use their concurrent analysis actions. Minimum compression eccentricities are 0.05t and 0.05L. In-plane magnification is 1.00.								
Combination	End	P kN	M22 kNm	M33 kNm	Design M22	Design M33	δ out	Util. %
LC-1.35DL	Bottom	-486.68	0		37	73	1	8.77
LC-1.35DL	Top	-397.56	0	-54.78	3.98	59.63	1	7.67
LC-1.2DL+1.5LL	Bottom	-507.3	0	756.44	5.07	756.44	1	48.96
LC-1.2DL+1.5LL	Top	-574.96	0	-411.33	5.75	411.33	1	27.95
LC-AUTO-1 Dead + 1 RS1 Component-peak envelope	Bottom	-330.2	0	121.23	3.3	121.23	1	11.58
LC-AUTO-1 Dead + 1 RS1 Component-peak envelope	Top	-266.34	0	-3.98	2.66	39.95	1	5.75
LC-AUTO-1 Dead + 0.6 Live + 1 RS1 Component-peak envelope	Bottom	-360.07	0	423.86	3.6	423.86	1	31.51
LC-AUTO-1 Dead + 0.6 Live + 1 RS1 Component-peak envelope	Top	-354.96	0			149.03	1	13.43
LC-AUTO-1 Dead - 1 RS1 Component-peak envelope	Bottom	-390.81	0	-121.48	3.91	121.48	1	11.62
LC-AUTO-1 Dead - 1 RS1 Component-peak envelope	Top	-322.65	0	-77.17	3.23	77.17	1	8.58
LC-AUTO-1 Dead + 0.6 Live - 1 RS1 Component-peak envelope	Bottom	-420.69	0	181.16	4.21	181.16	1	15.38
LC-AUTO-1 Dead + 0.6 Live - 1 RS1 Component-peak envelope	Top	-411.28	0	-222.22	4.11	222.22	1	17.99

Figure 3.1 – Extract from FEAkalc Design Memo for Piers

3.4. Separation of spectrum shear contributions

For each selected combination and member end, the shear reader retains the signed, already factored contribution from every response spectrum case. A case is recognised as a spectrum contribution by its analysis metadata. A name containing earthquake letters is not, by itself, the classification rule used to form this contribution.

$$V^E = \Sigma_{\text{spectrum}} \lambda_k V_{2,k}$$

If V is the original combination shear and Ω is the nominated factor, the amplified shear is:

$$V_{\text{amplified}} = V + (\Omega - 1)V^E$$

This expression adds amplification only to the spectrum contribution. It is equivalent to leaving the static part unchanged and multiplying the signed spectrum part by Ω. Multiple spectrum terms are summed with their factors before taking a magnitude.

The end design demand envelopes the original and amplified magnitudes. Retaining the original magnitude prevents a cancellation after amplification from reducing the design demand below the analysis value.

$$V_{\text{design,end}} = \max(|V|, |V_{\text{amplified}}|)$$

The factor Ω is an engineer input with Ω ≥ 1. It is not an automatically established capacity design factor or a demonstration that every seismic requirement of Section 14 has been satisfied. It does not multiply the pier axial force, either pier section moment, or the spandrel analysis moments used for longitudinal ties.

3.5. Two pier shear demand paths

The current desktop workflow supplies a separate shear action envelope when selected combinations include spectrum results. In that path the equation in Section 3.4 is used, and the section capacity based seismic multiplier is not applied again. A static row within the same run has V^E = 0 and retains its original shear.

A separate shear envelope is also supplied where the available section and shear action counts differ. Such a result must be read with its incomplete coverage qualification, rather than treated as a complete design of the selected combinations.

Where the combined section design is called without that separate shear list, the retained static earthquake path uses a multiplier based on the provided section nominal in plane moment capacity at that row axial load. The earthquake designation for this path is an explicit selection, not a case name inference. Chapter 5 gives the equation and its zero moment branch.

These are alternative demand paths. Multiplying the spectrum contribution by Ω and then multiplying the complete result again by the legacy capacity based factor would double count the implemented amplification and is not the FEAKalc calculation.

In-plane shear and horizontal reinforcement — AS 3600:2018	
Wall height H / length L / H/L	3,000 / 2,000 mm / 1.5
Concrete contribution V_{uc} (11.6.3)	505.96 kN
Provided horizontal reinforcement	N12 @ 350 mm EF; 646.27 mm ² /m total
Required / provided horizontal ratio	0.25% / 0.323%
Minimum / crack control (11.7)	0.25%; A1, Minor, restrained
Effective reinforcement ratio (11.6.4)	0.323%
Reinforcement contribution V_{us}	517.02 kN
Shear ϕ / web crushing ϕ	0.75 / 0.70
Web crushing $V_{u,max}$ / design limit	2,560 / 1,792 kN
Design capacity $\min[\phi(V_{uc}+V_{us}), 0.70V_{u,max}]$	767.24 kN
Governing shear	LC-1.2DL+1.5LL / Bottom; 49.66%, PASS

With $r = H/L$, $V_{uc} = (0.66 - 0.21r)/f_c(0.8L_t)$ for $r \leq 1$. For $r > 1$, use the lesser of that expression and $[0.05 + 0.1/(r - 1)]/f_c(0.8L_t)$, but not less than $0.17/f_c(0.8L_t)$. $V_{us} = \rho_w f_y s(0.8L_t)$; $V_{u,max} = 0.2f_c(0.8L_t)$. Required shear ratio = $\max[0, (V^*/\phi - V_{uc})/(f_y s 0.8L_t)]$. Calculations use N and mm; displayed forces are kN.

In-plane shear and horizontal minimum/crack-control reinforcement satisfied. The basic horizontal minimum is retained at 0.25%. For restrained walls exposure and crack-control ratios may govern. $\phi = 0.75$ assumes Class N fitments meeting 8.2.1.7; the engineer must detail them accordingly. Web crushing always uses $\phi = 0.70$.

Combination	End	Original V2 kN	Spectrum VE kN	Factor Ω	Amplified V2 kN	Design V kN	Shear util. %
LC-1.35DL	Bottom	-0.29	0	1	-0.29	0.29	0.04
LC-1.35DL	Top	0.75	0	1	0.75	0.75	0.1
LC-1.2DL+1.5LL	Bottom	381.04	0	1	381.04	381.04	49.66
LC-1.2DL+1.5LL	Top	-3.8	0	1	-3.8	3.8	0.5
LC-AUTO-1 Dead + 1 RS1	Bottom	43.09	43.3	1.5	64.74	64.74	8.44
LC-AUTO-1 Dead + 1 RS1	Top	0.89	0.33	1.5	1.06	1.06	0.14

Overstrength factor for shear, user need not to create load combos for this

Overstrength factor for shear, user need not to create load combos for this

Figure 3.2

3.6. Governing combinations and incomplete coverage

The governing section row is the row with the largest section utilisation for the selected arrangement. The governing shear row is determined independently. Boundary zone length and restraint category also have independent governing rows. Their names can differ without indicating an inconsistency.

The pier workflow retains available checks where another selected combination cannot be read, and records the missing coverage. An otherwise passing result with missing section coverage is PARTIAL. If no valid section action sets remain, the member reports an error. The server also checks that each supplied shear combination and end has a matching section action.

A partial result remains useful for diagnosing available demands, but its coverage must be completed before it is treated as a design of the full selected set. Missing or nonfinite shear cannot be silently interpreted as zero. Intentionally unloaded cases must be represented by valid explicit zero results, rather than by missing data.

4. Pier Axial Force and Bending Design

4.1. Section model

The pier is designed as a rectangular horizontal section of dimensions t by L . The longitudinal reinforcement consists of n bars on each of two equal face rows. The same reinforcement centre c is used from the thickness faces and from the length ends of the section idealisation. The arrangement is symmetric, allowing positive and negative bending to be assessed using the corresponding capacity magnitude.

$$A_{\text{bar}} = \pi d_{\text{p}}^2 / 4 \quad A_s = 2nA_{\text{bar}} \quad \rho_v = A_s / (Lt)$$

$$s_v = (L - 2c) / (n - 1)$$

This is a distributed two face wall arrangement, not an optimisation of concentrated boundary bars or unequal face reinforcement. The strain compatibility calculation uses the actual arrangement represented by these rows. An engineer who adopts a substantially different bar distribution must recheck the section capacities for that distribution.

4.2. Concrete and reinforcement properties

The engine uses a rectangular concrete compression block with strength dependent parameters. The parameters adopted by the inherited rectangular column routine are:

$$\alpha_1 = \min[0.85, \max(0.72, 1 - 0.003f'c)]$$

$$\alpha_2 = \max(0.67, 0.85 - 0.0015f'c)$$

$$\gamma = \max(0.67, 0.97 - 0.0025f'c)$$

The normal section interaction calculation uses α_2 and γ . The α_1 parameter participates in the squash load and the inherited high compression construction. At $f'c = 40$ MPa, these expressions give $\alpha_1 = 0.85$, $\alpha_2 = 0.79$ and $\gamma = 0.87$.

The adopted ultimate extreme concrete compressive strain is 0.003 and the reinforcing steel modulus is 200000 MPa. Reinforcement is elastic perfectly plastic, with tensile and compressive stresses limited to its yield strength. The pier input validation limits both longitudinal and transverse yield strengths to 500 MPa for the implemented Section 11 wall procedure.

4.3. Strain compatibility

Plane sections are assumed to remain plane. For a neutral axis depth a measured from the compressed face and a bar at depth y_i , its strain follows the linear section strain distribution. Compression is positive within the section calculation.

$$\epsilon_{s,i} = 0.003(a - y_i) / a$$

$$f_{s,i} = \max[-f_y, \min(f_y, E_s \epsilon_{s,i})]$$

The neutral axis is commonly expressed as $a = k_u d$, where d is the effective depth for the direction being considered. The weak direction uses $d_{22} = t - c$; the strong direction uses $d_{33} = L - c$. Varying k_u varies the compression block and every reinforcement strain, generating a family of axial force and moment capacities.

Concrete tension is omitted from the ultimate resistance model. The reinforcement provides the tensile resultant. In compression, the inherited engine subtracts the concrete block stress from the compressive steel force to allow for the concrete displaced by the reinforcement, as represented in that engine. The manual describes this numerical model rather than substituting a different layered section analysis.

4.4. Force and moment equilibrium

For bending about one axis, let b be the section breadth perpendicular to the strain gradient. The normal rectangular stress block has depth γa and compressive force C_c . With stresses in MPa and dimensions in mm, the force is in N.

$$C_c = \alpha_2 f'_c b \gamma a$$

$$F_{s,i} = A_{s,i} f_{s,i} \text{ with the engine compression displacement correction}$$

$$N_u = C_c + \Sigma F_{s,i}$$

The moment is calculated about the section centroid by summing the moments of the concrete and reinforcement resultants. For a section depth h in the bending direction, the concrete resultant acts at $\gamma a/2$ from the compressed face.

$$M_u = C_c(h/2 - \gamma a/2) + \Sigma[F_{s,i}(h/2 - y_i)]$$

Division by 1000 converts N to kN, and division by 1000000 converts N mm to kN m. Both directions are calculated independently with the appropriate bar layers and section dimensions. The result is a weak axis N M22 curve and a strong axis N M33 curve.

4.5. Axial endpoints and capacity reduction

The inherited design squash load is based on the net concrete area and longitudinal steel area. The compression reduction factor at this endpoint is 0.60.

$$N_{uo} = \alpha_1 f'_c (A - A_s) + A_s f_y$$

$$N_{\text{compression,design}} = 0.60 N_{uo}$$

For the pier wrapper, a pure tension endpoint is added explicitly. Its uniaxial design capacity is:

$$N_{\text{tension,design}} = 0.85 A_s f_y$$

The interaction curves retain the point dependent ϕ values generated by the inherited section routine. The compression controlled region starts at 0.60. In the normal sampled curve, k_u values below approximately 0.54 increase ϕ in increments of 0.25/35, capped at 0.85. The curve sampling proceeds in k_u increments of 0.01 and includes an inherited straight transition near the compression end.

Consequently the delivered capacity is the polygonal numerical envelope produced by that routine. It should not be confused with an exact analytic interaction curve or a newly derived continuous ϕ expression. Rounding and discretisation can cause small differences from another section program.

4.6. Capacity at the applied axial force

The pier routine intersects every segment of the interaction curve with the applied axial force N^* . It does not use the older radial utilisation search of the frame column routine. Where a segment joins points (M_a, N_a) and (M_b, N_b) , the interpolated capacity is:

$$M_{\text{cap}}(N^*) = M_a + (M_b - M_a)(N^* - N_a)/(N_b - N_a)$$

Only segments whose axial range contains N^* participate. For a horizontal segment, the larger endpoint moment is retained. Where more than one segment intersects the axial level, the largest nonnegative moment capacity is adopted. If the axial level is outside the available envelope or no positive required moment capacity exists, the row cannot pass.

For the nominal moment required by the legacy pier seismic shear branch, each design curve point is first divided by its stored ϕ , then intersected at the applied axial force. For biaxial resistance, the point reduction

described in Section 4.9 is made before intersection. In both cases the axial ordinate changes with the point scaling as well as the moment ordinate.

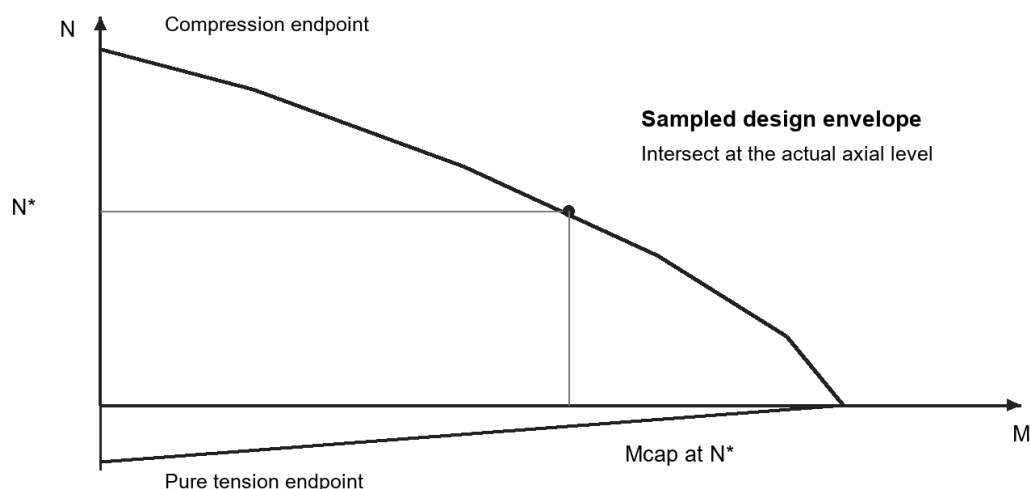


Figure 4.1- Interaction curve intersection at the applied axial force

4.7. Minimum eccentricities

For compressive axial force, each direction retains a minimum eccentricity equal to five percent of the corresponding overall section dimension. The adopted moments before optional magnification are:

$$M_{22,base} = \max[|M_{22}|, \max(0, N^*) \times 0.05t/1000]$$

$$M_{33,base} = \max[|M_{33}|, \max(0, N^*) \times 0.05L/1000]$$

These equations use N^* in kN and t and L in mm, giving moments in kN m. For tension, the compression eccentricity terms are zero. The adopted design moments are magnitudes; the original signed analysis moments are retained in the report.

Even when the analysis M_{22} is zero, a compressive row can therefore have nonzero adopted moments about both axes and require the biaxial assessment. Conversely, a genuinely uniaxial row with no compression minimum in the other direction is not assessed against a fictitious biaxial diagram.

4.8. Optional out of plane buckling

The global option Design wall for out of plane buckling controls the weak direction moment magnifier. When it is unselected, both magnifiers are exactly 1.00. Wall height does not activate magnification automatically. The designer must assess actual restraint, including floors, return walls and buttressing, when choosing this option.

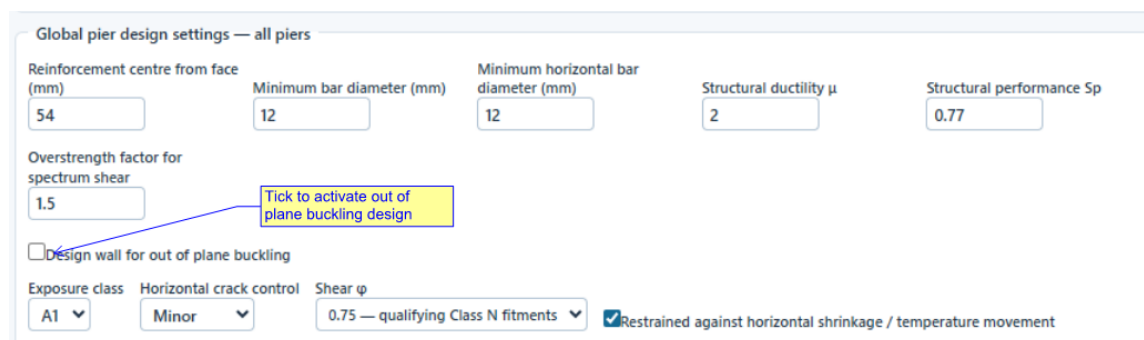


Figure 4.2

When the option is selected and N^* is compressive, the routine uses the specified out of plane effective length l_e and sustained load ratio β_d . It adopts a conservative moment gradient coefficient $k_m = 1$ rather than inferring a gradient from the synthetic end moment pair supplied for a section row.

$$N_c = \pi^2 [182 d_{22} (\phi M_{ub})] / [l_e^2 (1 + \beta_d)]$$

In this expression ϕM_{ub} is the weak direction base moment evaluated by the inherited section routine at $k_u = 0.545$ with $\phi = 0.60$. Use N and N mm consistently in this form of the equation. The engine converts its stored $kN\ m$ moment to N mm before evaluating the critical load.

$$\delta_{22} = \max[1, 1/(1 - N^*/N_c)] \quad \text{for } 0 < N^* < N_c$$

$$M_{22,design} = \delta_{22} M_{22,base} \quad M_{33,design} = M_{33,base}$$

Compression at or above N_c fails the stability check; a negative denominator is not accepted as a beneficial reduction. Tensile rows are not magnified. In plane magnification remains 1.00. This member calculation does not replace any required global second order or sway analysis.

4.9. Uniaxial and biaxial checks

At each applied axial force the routine obtains the uniaxial capacities $M_{22,cap}$ and $M_{33,cap}$. Their utilisations are the corresponding adopted moments divided by these capacities. The axial endpoint comparison is also included.

$$U_{22} = M_{22,design} / M_{22,cap} \quad U_{33} = M_{33,design} / M_{33,cap}$$

Where both adopted design moments are nonzero, the biaxial check limits the curve point reduction factor to 0.65. A point with stored $\phi > 0.65$ has both its moment and axial ordinates multiplied by $0.65/\phi$. A point with $\phi \leq 0.65$ is unchanged. The pure tension endpoint for this reduced envelope is $0.65 A_s f_y$.

The capacities $M_{22,b}$ and $M_{33,b}$ are obtained by intersecting the reduced envelopes at the actual N^* . The exponent is based on the nominal squash capacity N_{uo} :

$$\alpha = \min[2, \max(1, 0.7 + 1.7 N^* / (0.65 N_{uo}))]$$

$$U_{biaxial} = (M_{22,design} / M_{22,b})^\alpha + (M_{33,design} / M_{33,b})^\alpha$$

The biaxial expression must not exceed 1.00. The section utilisation is the maximum of the applicable uniaxial, biaxial and axial comparisons, with instability causing failure. When a row is uniaxial, the biaxial check is marked inapplicable and its reduced envelope is omitted from that row memo.

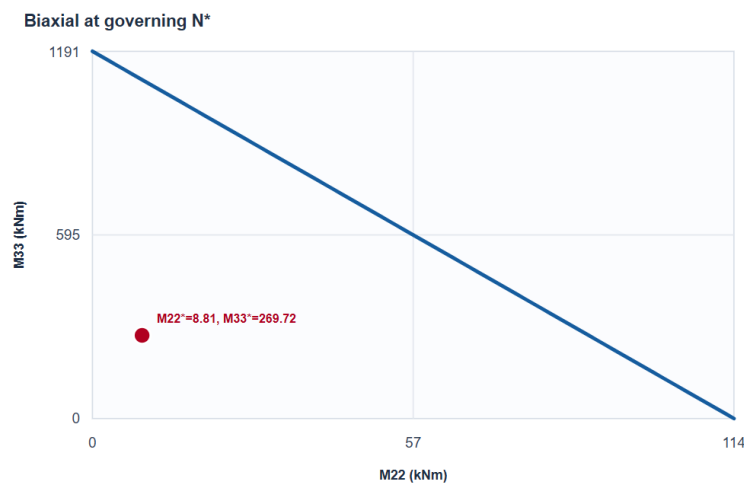


Figure 4.3 -Bi-Axial Check

4.10. Longitudinal reinforcement search

The search starts with standard diameters 12, 16, 20, 24, 28, 32 and 40 mm, excluding diameters smaller than the nominated global minimum. For each diameter it considers equal bars per face, subject to the two face geometry and spacing checks. At least two bars per face are required, and the search is capped at 200 bars per face.

$$0.0025 \leq \rho_v \leq 0.04$$

$$s_v \leq \min(2.5t, 350 \text{ mm}) \quad s_v - d_\beta \geq 3d_\beta$$

The latter means the centre spacing must be at least $4d_\beta$. The engine also checks $c > d_\beta/2$ and $t - 2c \geq 4d_\beta$ for the two face arrangement. These are the program fit rules; aggregate size, practical placing tolerances and the final fitment cage must still be assessed.

For every longitudinal candidate, every available section row is checked and the corresponding shear design is calculated. This is particularly important for $H/L \leq 1$, where vertical reinforcement limits the effective shear reinforcement ratio. A candidate that passes flexure but fails shear is not accepted as the common design.

Among passing candidates the routine selects the smallest actual longitudinal steel area within the search. This is an optimisation of the stated discrete arrangement, not a proof of the globally least cost reinforcement layout. If no candidate passes, the reported best available candidate remains a failure; if no arrangement fits at all, an error is returned.

The section capacity engine uses the total steel area rounded to the nearest whole square millimetre and redistributes that area among its layers. The displayed steel area and reinforcement percentage use the actual circular bar areas. Small differences between a capacity calculation and the displayed area are therefore expected.

5. Pier In Plane Shear Design

5.1. Wall height and effective shear area

Pier shear follows the implemented wall provisions of AS 3600 Section 11, with the demand treatment described in Chapter 3. The wall height H used in the aspect ratio is an engineer input. It is initially the full labelled height, but a storey label is not automatically evidence that this is the appropriate structural wall height for the shear equations.

$$r = H/L \quad A_v = 0.8L_t$$

The same section length and thickness used for pier flexure enter the shear area. H must be reviewed for the actual wall system and intended code application. The labelled height, shear height H , optional buckling effective length, and overall building height used for boundary classification serve different purposes and must not be interchanged.

5.2. Concrete contribution

The nominal concrete contribution V_{uc} depends on the aspect ratio and square root of concrete strength. For $r \leq 1$, the coefficient is $0.66 - 0.21r$. For $r > 1$, the engine takes the bounded lesser expression shown below.

$$k_v = 0.66 - 0.21r \quad \text{for } r \leq 1$$

$$k_v = \max[0.17, \min(0.66 - 0.21r, 0.05 + 0.1/(r - 1))] \quad \text{for } r > 1$$

$$V_{uc} = k_v \sqrt{f'_c} A_v / 1000 \quad \text{in kN}$$

The $r = 1$ case is evaluated on the first branch, avoiding division by zero. Both sides approach 0.45 at that boundary. For larger aspect ratios the lower bound prevents the adopted coefficient falling below 0.17. The branch selection and bounds must be applied before calculating reinforcement demand.

5.3. Reinforcement contribution and squat walls

The effective shear reinforcement ratio depends on the relative height and length of the wall. For a squat wall, both reinforcement directions participate and the lesser ratio limits the steel contribution. For a taller wall, the horizontal ratio is used.

$$\rho_{eff} = \min(\rho_v, \rho_h) \quad \text{for } H/L \leq 1$$

$$\rho_{eff} = \rho_h \quad \text{for } H/L > 1$$

$$V_{us} = \rho_{eff} f_{ys} A_v / 1000 \quad \text{in kN}$$

The horizontal ratio represents the total reinforcement on both faces divided by the corresponding concrete strip area. It is not the ratio on one face only. Likewise, ρ_v is the total longitudinal steel area on both faces divided by L_t .

Increasing horizontal reinforcement cannot compensate indefinitely for insufficient vertical steel in a squat wall. The unified search addresses this by checking the shear result for each longitudinal candidate. The designer should still inspect which direction is limiting, particularly where the section reinforcement is governed by shear rather than by flexural interaction.

5.4. Strength reduction and web crushing

The nominal web crushing capacity and the final design shear capacity are:

$$V_{u,max} = 0.2f'_c A_v / 1000 \quad \text{in kN}$$

$$V_{\text{capacity}} = \min[\phi_v(V_{uc} + V_{us}), 0.70V_{u,\text{max}}]$$

The general shear factor ϕ_v is 0.70 by default. The optional 0.75 value assumes qualifying Class N fitments meeting the applicable detailing conditions of Clause 8.2.1.7. Selection of the higher value is an engineering detailing assumption, not a classification established by the program from the selected diameter.

The web crushing factor remains 0.70 even when the general shear factor is 0.75. If the design demand exceeds $0.70V_{u,\text{max}}$, adding reinforcement alone cannot produce a passing section. Revise the dimensions, concrete strength or structural demand and then redesign.

$$U_{\text{shear}} = V_{\text{design}}/V_{\text{capacity}} \quad U_{\text{crushing}} = V_{\text{design}}/(0.70V_{u,\text{max}})$$

The routine reports both utilisations and includes shear in the overall pier utilisation. A reinforcement arrangement that does not fit is explicitly failed, even if a theoretical area could satisfy the strength equation.

5.5. Required horizontal ratio

For each action row the strength equation is rearranged to obtain the horizontal ratio demand before discrete bar selection. The result is bounded below by zero and then enveloped with the minimum and crack control ratios.

$$\rho_{\text{strength}} = \max[0, (V_{\text{design}}/\phi_v - V_{uc})1000/(f_{ys}A_v)]$$

$$\rho_{h, \text{required}} = \max[\rho_{\text{min}}, \text{maxrows}(\rho_{\text{strength}})]$$

For $H/L \leq 1$ this calculation establishes the horizontal requirement but does not remove the independent restriction imposed by ϕ_v . The final capacity is always recalculated using the lesser direction ratio. This final check is necessary even after the horizontal strength ratio has been met.

5.6. Minimum steel and crack control

The base horizontal ratio is 0.0025. The program does not take conditional reductions below this value. Where horizontal shrinkage and temperature movement is restrained, the exposure and nominated crack control level can increase the minimum.

Exposure	Horizontal movement	Minor	Moderate	Strong
A1 or A2	Restrained	0.0025	0.0035	0.0060
B1 B2 C1 or C2	Restrained	0.0060	0.0060	0.0060
Any supported class	Unrestrained	0.0025	0.0025	0.0025

For the conditional $\phi_v = 0.75$ selection, the minimum also includes the additional shear reinforcement expression:

$$\rho_{\text{min}} \geq 0.08\sqrt{f'_c}/f_{ys}$$

The exposure, restraint and crack control inputs must represent the actual conditions. The selected ratios are implemented minimum steel rules; they do not calculate crack widths, shrinkage strains or restraint forces for a particular wall length and construction sequence.

5.7. Horizontal bar arrangement

Horizontal reinforcement is selected on both faces from diameters 10, 12, 16, 20, 24, 28, 32 and 40 mm, excluding sizes below the nominated global minimum. For diameter d_h and spacing s_h , the provided total ratio and total steel per metre are:

$$\rho_h = 2(\pi d_h^2 / 4) / (s_h t)$$

$$A_{sh, \text{ per metre}} = \rho_h t \times 1000$$

The maximum spacing is the least of $2.5t$, 350 mm and the spacing that supplies the required ratio. The trial spacing is rounded downward to a 25 mm increment. It must also provide at least $3d_h$ clear spacing, equivalent to $s_h \geq 4d_h$.

$$s_{h, \text{ trial}} = 25 \text{ floor} \{ \min[2.5t, 350, 2A_{bar} / (t\rho_{h, \text{ required}})] / 25 \}$$

The fit checks require the reinforcement centre to leave room for a horizontal bar outside the vertical bar and adequate spacing between the two face assemblies. In the implemented inequalities, $c > d_v/2 + d_h$ and $t - 2c + d_v \geq 3d_h$. These conditions explain why increasing horizontal diameter can make an otherwise adequate area infeasible.

Among fitting horizontal options the routine selects the smallest provided horizontal ratio. The returned schedule is expressed as a diameter and spacing EF, meaning each face. A total area per metre reported beside that schedule includes both faces and must not be assigned again to each face.

5.8. Seismic demand without the separate spectrum list

The retained combined section path uses the nominal in plane moment strength of the provided section at the action row axial force. For a row explicitly designated earthquake, with $\mu > 1$ and nonzero analysis M33, its multiplier is:

$$\omega_{\text{legacy}} = \max[1, \min(1.6M_{w, \text{ nom}} / |M_{33}|, \mu / Sp)]$$

$$V_{\text{amplified}} = \omega_{\text{legacy}} V$$

If $|M_{33}|$ is effectively zero, the finite μ/Sp branch is used and bounded below by 1. A non earthquake row, or a row with $\mu = 1$, has multiplier 1. The shear demand still includes the original magnitude. The μ/Sp values default to 2/0.77 but remain engineering inputs.

This path depends on the provided longitudinal section and therefore can change during the reinforcement search. It applies only when the combined calculation is used without the separate shear component envelope. For a spectrum component run, the nominated Ω rule of Chapter 3 takes its place; the two are not cumulative.

5.9. Checked stations and scope

The shear checks are made at the available Top and Bottom stations. The implementation does not apply the permitted near base reduction of Clause 11.6.1. It also does not infer an internal maximum between those stations.

Out of plane shear V3 and torsion are outside the current pier shear calculation. The issued wall shear equations are used for tensile as well as compressive action rows; the different routing and cyclic reductions proposed in later draft provisions are not introduced. The designer must assess applicability to the actual load and ductility regime.

6. Pier Boundary Zone Screening

6.1. Purpose and independent status

The boundary calculation identifies edge regions requiring restraint under the implemented screening of Section 14.6. It uses gross rectangular elastic stresses from the original signed analysis actions. It is separate from the nonlinear ultimate section interaction calculation and does not use its absolute moments, minimum eccentricities or magnifiers.

The output is an extent and restraint category, not a designed fitment cage. It also reports a whole wall core confinement requirement where applicable. A pier can pass its section and shear checks while requiring boundary restraint; the boundary requirement is not converted into an additional section utilisation ratio.

6.2. Gross elastic edge stresses

Take x along local 2, with $x = -L/2$ at Edge 1 and $x = +L/2$ at Edge 2. Compression is positive. The more compressed thickness face is used at each edge, which introduces the magnitude of the M_{22} contribution.

$$\sigma(x) = -P \times 1000/A + M_{33} \times 10^6 x/I_{33} + |M_{22}| \times 10^6 (t/2)/I_{22}$$

The two extreme edge stresses in MPa become:

$$\sigma_1 = -1000P/(Lt) - 6 \times 10^6 M_{33}/(tL^2) + |6 \times 10^6 M_{22}/(Lt^2)|$$

$$\sigma_2 = -1000P/(Lt) + 6 \times 10^6 M_{33}/(tL^2) + |6 \times 10^6 M_{22}/(Lt^2)|$$

A positive M_{33} increases compression toward Edge 2. Moment reversal can therefore transfer the boundary requirement to the opposite edge. Out of plane bending can increase the controlling face compression at both edges even where the in plane moment is unchanged.

These stresses are an elastic screening device. They are not the actual cracked reinforced concrete stress distribution at ultimate resistance. Spectrum rows retain the qualification that their inputs are component peaks, rather than a demonstrated simultaneous stress state.

6.3. Zone length at the threshold

The threshold for the edge zone calculation is $q = 0.15f'_c$. The routine assumes a linear stress variation along L on the controlling thickness face. For a selected edge stress σ_e and the opposite stress σ_o , its length is zero where $\sigma_e \leq q$.

If $\sigma_e > q$ and $\sigma_o \geq q$, the full length is within the zone. If the threshold lies between the edge values, the analytical crossing gives:

$$l_{\text{zone}} = L(\sigma_e - q)/(\sigma_e - \sigma_o)$$

The result is bounded between zero and L . At an exactly threshold edge the implemented strict trigger does not create a zone. If both edges are above the threshold, both edge outputs can report the complete length. Overlapping outputs do not imply two separate reinforcement quantities over the same material; they describe the required restraint extent.

Gross elastic stress on the more compressed face

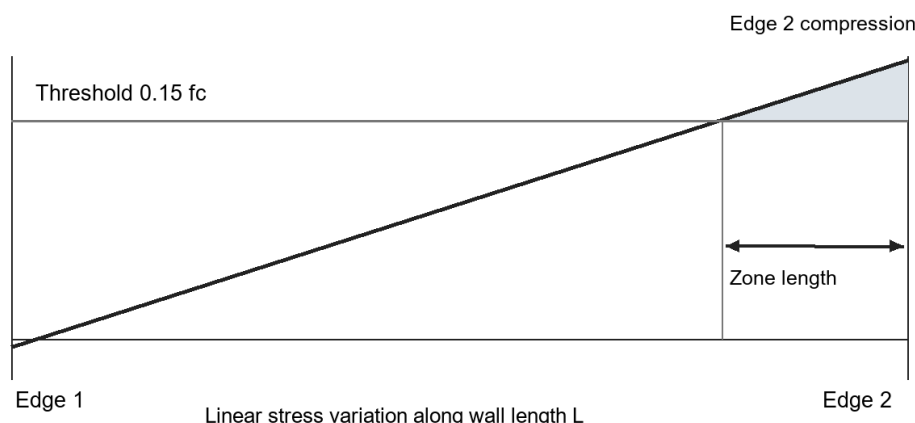


Figure 6.1 - Linear edge stress screening and threshold crossing

6.4. Building height classification

For the implemented screening, overall building height is the vertical extent of all model shells and frame members, including unlabelled objects. At the engineer adopted rule, a vertical extent greater than 12 m selects the more than four storey branch; at or below 12 m it selects the low rise branch. A small numerical tolerance suppresses floating point changes at exactly 12 m.

This is a program classification proxy adopted for this project. It is not the wording of a code definition of storey count. The designer must confirm that the branch represents the actual building. An unrelated model object extending above or below the building can change this proxy and should be reviewed.

The building height is independent of the pier labelled height and of H used in shear. Changes elsewhere in the model can invalidate the previous boundary classification even when the designed pier itself has not changed.

6.5. Restraint categories

For $\mu \leq 1$ the program reports REVIEW REQUIRED and refers the designer to the nonductile wall assessment under Clause 11.7.4. It does not automatically apply the ductile wall stress trigger to those walls.

For the ductile screening branches, a required boundary zone is assigned the category appropriate to the ductility and building classification. The selection is summarised below.

Ductility and classification	Additional condition	Restraint category where a zone is required
Limited ductile low rise	μ greater than 1 and no greater than 2	Open U bars or closed ties to 14.6.2.2
Limited ductile above 12 m proxy	Maximum edge compression no greater than $0.20f_c$	Closed fitments to 14.6.2.3 and 10.7.4
Limited ductile above 12 m proxy	Maximum edge compression greater than $0.20f_c$	Closed fitments to 14.5.4
Moderately ductile	μ greater than 2	Closed fitments to 14.5.4

For the low rise deemed arrangement, every triggered zone is extended to at least 1.5t, limited to the wall length. This implements the adopted extent associated with Figure 14.6.2.2. It does not calculate the U bar or tie area.

$$l_{\text{low rise}} = \min[L, \max(l_{\text{zone}}, 1.5t)] \quad \text{where } l_{\text{zone}} > 0$$

For the taller limited ductile branch, the enhanced category is determined from the larger edge compression in that action row. It applies to required edge zones in that row. For moderate ductility, the enhanced category does not depend on exceeding 0.20f_c.

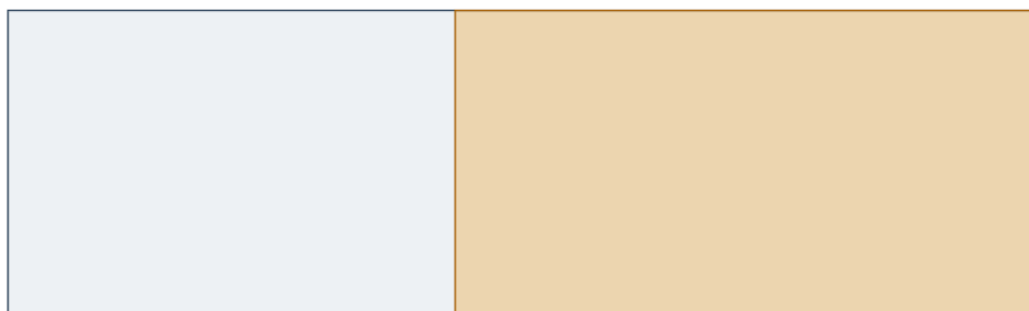
Boundary-zone screening — REQUIRED

Building classification uses the engineer's rule: model vertical extent > 12 m means more than four storeys. Existing vertical reinforcement is assumed not already restrained to 10.7.4. Edge 1 = -local 2; Edge 2 = +local 2. Signed analysis N/M and gross rectangular properties are used, including the more compressed thickness face; section-design minimum eccentricities and moment magnifiers are not substituted. Only supplied top/bottom stations are screened. Zone lengths are enveloped for adoption over the labelled pier height; intermediate sections are not verified. Fitment sizes, spacing, anchorage and existing restraint are for the designer. All selected ULS combinations are screened; spectrum rows retain their component-peak-envelope qualification. The 0.3f_c axial-force provisions are not treated as local stress triggers. Critical tension reinforcement zones under 14.6.7 remain a separate check.

Model height 39 m: more than four storeys by the adopted 12 m rule. No fitment schedule is designed.

Edge 1 (-local 2): 0 mm

Edge 2 (+local 2): 1,135 mm



Adopted end-zone envelope over labelled pier; top/bottom checked only.

Shading indicates required zones, not a fitment arrangement.

Edge	Zone mm	Restraint category	Length governs	Category governs	Max compression MPa
1 (-local 2)	0	Not required by this screening			3.246
2 (+local 2)	1,135	Closed fitments to 14.5.4	LC-1.2DL+1.5LL Bottom	LC-1.2DL+1.5LL Bottom	10.455

Compression-positive stress: $\sigma(x) = -P/A + M33 \cdot x/I33 \pm M22(t/2)/I22$; the more compressed thickness face governs. $A = Lt$, $I33 = tL^3/12$, $I22 = Lt^3/12$. Lengths follow the $\sigma > 0.15f_c$ crossing. For the low-rise deemed arrangement, adopted lengths are at least 1.5t (Figure 14.6.2.2). Above 0.20f_c, taller limited-ductile walls require 14.5.4 restraint; moderately ductile boundary elements require it regardless of stress. The strongest category is adopted throughout each enveloped edge zone.

Figure 6.2 – FEAkalc Design Memo (Boundary Zone Check)

6.6. Enveloping extent and category

Zone lengths are calculated for both edges for every available selected Top and Bottom section row. The largest length is retained independently at each edge. Separately, the strongest applicable restraint category encountered in a row with a nonzero zone is retained for that edge.

The governing length row can therefore differ from the governing category row. The adopted result applies the strongest category throughout the enveloped length. Recording both governing rows allows the designer to see why a relatively long low stress zone may receive a stronger restraint arrangement triggered by another combination.

Displayed millimetre lengths round upward. Stored and exported lengths retain their calculated precision. The shaded elevation zone shows the adopted envelope over the labelled pier height, but only the supplied Top and Bottom stations have actually been screened. The shading does not establish the sufficiency of intermediate sections.

6.7. High strength concrete and remaining assessments

For ductile walls with $f'_c > 50$ MPa, whole wall core confinement to Clause 14.5.4 is independently flagged under 14.6.3. It is not limited to the edge lengths obtained from the $0.15f'_c$ crossing. In the nonductile branch the review status remains and the designer must make the applicable assessment separately.

The routine assumes that existing vertical reinforcement has not already been restrained to Clause 10.7.4. It does not inspect an existing cage. Fitment diameter, spacing, cross tie layout, anchorage, lap locations and restraint effectiveness remain design tasks.

The $0.3f'_c$ axial force provisions are not used as substitutes for the local $0.15f'_c$ and $0.20f'_c$ stress criteria. Separate axial force confinement requirements of Sections 10 and 11 and critical tension reinforcement zones under 14.6.7 remain to be assessed. NOT REQUIRED BY SCREENING means only that this screening did not produce a requirement.

7. Spandrel Coupling Beam Design

7.1. Adopted load path

The spandrel procedure represents the coupling beam by an equilibrium strut and tie model. Concrete diagonal struts carry compression, steel ties carry tension, and nodal regions transfer the forces between these components and the adjoining wall. It is a discontinuity region idealisation intended to explain the implemented shear and nodal calculation.

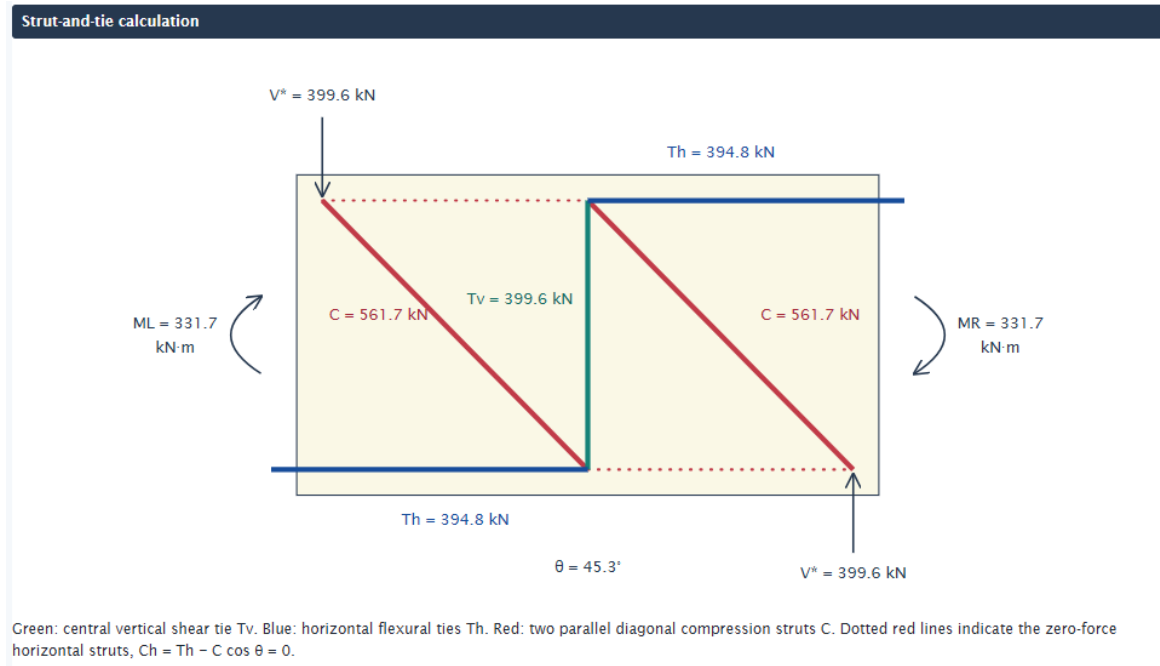


Figure 7.1 – FEAKalc Design Model for Coupling Beams – Extract from Design Memo

Two related but distinct calculations are made. The shear mechanism uses the enveloped design shear and equal equivalent end moments. The required longitudinal tie reinforcement uses the original analysis end moments. Keeping these calculations distinct prevents an artificial relationship between analysis moment and amplified shear from being imposed on the longitudinal tie demand.

The program does not automatically select between a conventional beam, diagonal reinforcement, and a fully detailed seismic coupling beam system. The designer must determine whether the adopted mechanism and reinforcement are appropriate for the span to depth ratio, ductility demand and connection details.

7.2. Input geometry and validation

The design inputs are concrete strength f'_c ; longitudinal and vertical tie yield strengths f_y and f_{ys} ; clear span l ; depth D ; thickness t ; edge node width w_e ; midspan node width w_m ; reinforcement centre c ; and nominated shear factor Ω . Dimensions are in mm and strengths in MPa.

$$z = D - 2c \quad L_w = l + w_e$$

The reinforcement centres must leave z positive. The edge node width must be less than half the clear span minus 100 mm, and the midspan node width must not exceed the clear span. All material, dimensional and factor inputs must be finite and positive, with $\Omega \geq 1$.

The working span L_w includes half an edge node width at each end. It is used to determine the shear model geometry. The denominator for distributed vertical tie steel remains the clear span l . Interchanging these two lengths changes the reinforcement demand and is not the implemented calculation.

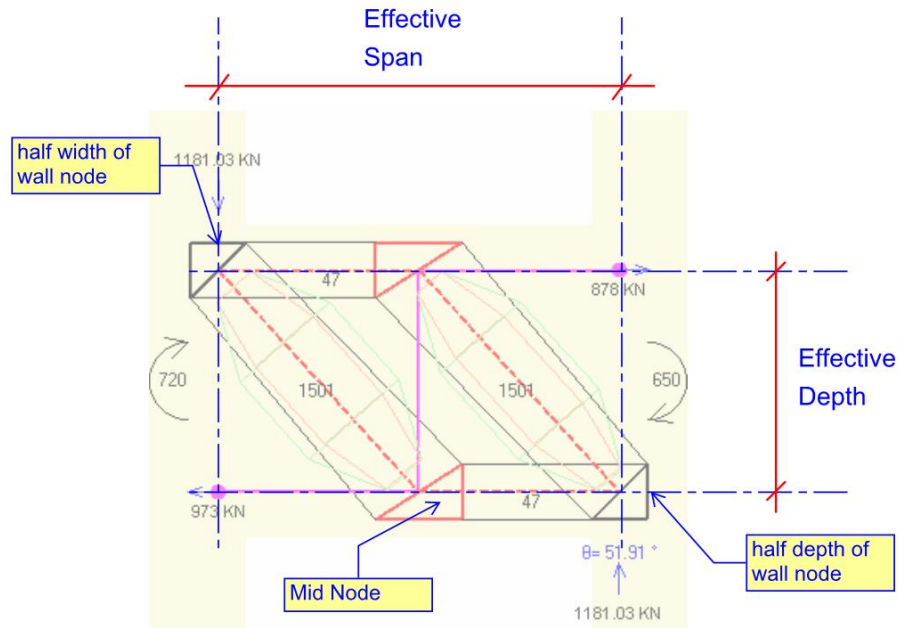


Figure 7.2 – Span and Depth of STM Coupling Beam in FEAkalc

7.3. End action envelopes

For every selected combination, both Left and Right V2 and M3 are read. The spectrum contribution at each end is retained with its sign and combination factor. At each end the original and spectrum amplified shear magnitudes are compared as described in Chapter 3.

$$V_{d,combo} = \max[|V_L|, |V_R|, |V_L + (\Omega - 1)V^E_L|, |V_R + (\Omega - 1)V^E_R|]$$

$$V_d = \text{maxselected combinations}(V_{d,combo})$$

The shear governing combination is the one that produces V_d . Longitudinal reinforcement is independently enveloped from the original left and right moments. Unequal end moments and zero shear with nonzero moment are therefore retained correctly in the longitudinal calculation.

7.4. Equal moment shear mechanism

For the shear model only, the equivalent end moment magnitude at each end is $V_d L_w / 2$. The end moments act in opposite senses.

Two parallel diagonal struts connect the upper left node to the lower midspan node and the upper midspan node to the lower right node. Each strut has rise z and horizontal run $L_w / 2$, giving the angle:

$$M_{eq} = V_d L_w / 2000 \quad \text{in kN m}$$

$$\theta = \text{atan}(2z / L_w)$$

Each diagonal strut carries compression C . The central vertical tie carries T_v and each active horizontal tie carries T_h :

$$T_v = V_d \quad C = V_d / \sin \theta$$

$$T_h = M_{eq} \times 1000 / z = V_d L_w / (2z)$$

Each strut has horizontal component $C \cos \theta = V_d \cot \theta = T_h$.

The top-left and bottom-right horizontal struts have $Ch = T_h - C \cos \theta = 0$ in this equal moment model. They are shown dotted in the sketch.

The equivalent moments are not replacements for the finite element moments used in the longitudinal tie design. Nor do they establish the actual moment distribution between the two recovered ends. They create the specified equilibrium shear model for calculating strut forces, vertical ties and node stresses.

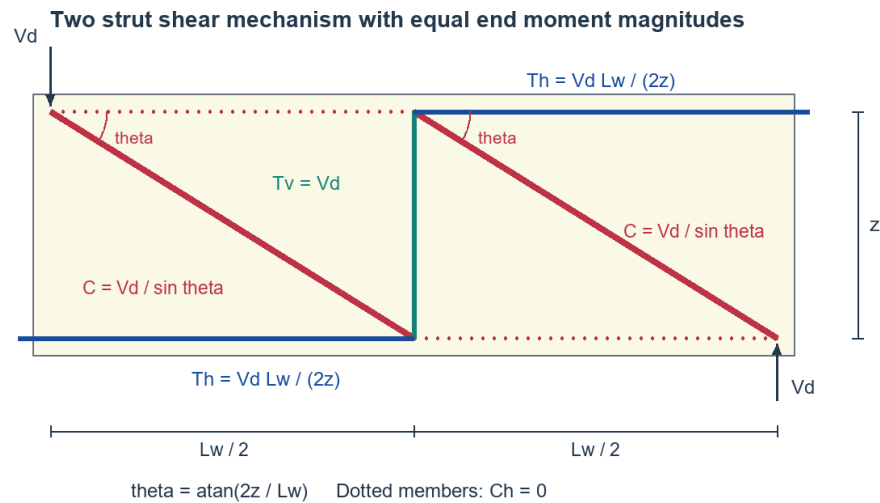


Figure 7.3 - Equal moment spandrel shear mechanism and principal forces

7.5. Vertical tie reinforcement

The vertical steel tie is designed with $\phi_t = 0.85$. The total steel assigned to that tie and the required steel per unit clear span are:

$$A_v = V_d \times 1000 / (0.85f_{ys})$$

$$A_v/s = V_d \times 1000 / (0.85f_{ys}l)$$

A_v is in mm^2 and A_v/s is in mm^2/mm . Multiply the latter by 1000 for total mm^2/m . The result is the total steel crossing the idealised vertical tie, not an automatically chosen diameter and spacing on each face.

For a proposed set of effective vertical legs, the provided area per unit length must be compared on the same total area basis. The number of legs and their anchorage must be established from the detail. A horizontal distribution of ties with the correct area does not by itself prove that every tie can develop its force into the nodal zones.

7.6. Longitudinal tie reinforcement from analysis moments

At each end of each selected combination, the longitudinal tie force is the absolute original analysis moment divided by the truss depth. Its required area uses the same steel tension tie reduction factor 0.85.

$$T_{\text{long, end}} = |M_{3, \text{end}}| \times 1000 / z \quad \text{in kN}$$

$$A_{s, \text{end}} = |M_{3, \text{end}}| \times 10^6 / (0.85f_y z)$$

The program returns separate left and right area envelopes and their maximum. It also identifies a governing longitudinal reinforcement combination, which may differ from the shear governing combination. The end area envelope must be interpreted with the signed action rows when deciding the upper and lower bar arrangements under reversal.

Ω is not a divisor in this calculation. Longitudinal moments are neither reduced by overstrength nor reconstructed from the amplified shear. The equal moment shear tie T_h and the analysis moment tie T_{long} can differ; both are reported so that the designer can assess compatibility of the chosen reinforcement with the complete force path.

This is a tie area calculation, not a general axial force and flexural interaction solution for the spandrel. Axial interaction, minimum longitudinal steel, development into the wall and the suitability of the adopted lever arm require separate review.

7.7. Nodal geometry

Three active nodal checks are calculated: Midspan CTT, Edge CCT and Edge CCC. In this nomenclature C denotes a compression component and T a tension component meeting at the node. The opposite midspan node has the same demand in the adopted equal moment model.

The midspan node uses width w_m ; both edge nodes use width w_e . Every active node has height $h = 2c$ and through thickness t . For node width w , its diagonal face length and orientation are:

$$q_{\text{node}} = \sqrt{(w^2 + h^2)} \quad \eta = \text{atan}(h/w)$$

The angle ψ between the strut force and the inclined face normal is $|\pi/2 - \theta - \eta|$ for the CTT and CCT checks. For the CCC check it is $|\theta - \eta|$, representing the orientation of that opposite node in the beam mechanism.

Increasing a node dimension changes both its direct bearing area and its inclined face orientation. It should therefore be chosen from a feasible anchorage and load spreading region, rather than increased solely to obtain a passing stress.

7.8. Applied nodal stresses

For the equal moment model, the force on the vertical face is T_h and the force on the horizontal face is V_d . Their direct stresses are:

$$\sigma_{\text{bearing}} = V_d \times 1000 / (tw)$$

$$\sigma_{\text{side}} = T_h \times 1000 / (th)$$

The inclined face stress is obtained from the strut force and the projected effective area. The transformed normal and shear components used in the check are:

$$\sigma_{\text{inclined}} = C \times 1000 / (tq_{\text{node}} \cos\psi)$$

$$\sigma_n = \sigma_{\text{inclined}} \cos^2\psi \quad \tau = \sigma_{\text{inclined}} \sin\psi \cos\psi$$

Set σ_{flat} to the greater of σ_{bearing} and σ_{side} . The engine obtains its controlling compressive stress from the following stress circle construction:

$$c_\sigma = (\sigma_n + \sigma_{\text{flat}}) / 2 - \tau^2 / [2(\sigma_{\text{flat}} - \sigma_n)]$$

$$\sigma_{\text{node}} = c_\sigma + \sqrt{[(c_\sigma - \sigma_n)^2 + \tau^2]}$$

If τ is effectively zero, σ_{node} is simply the greater of σ_n and σ_{flat} . If the denominator is effectively zero while the shear component is nonzero, the engine rejects the singular geometry. It does not return a spurious finite nodal capacity check.

These equations describe the program stress construction. σ_{node} is not assumed to equal only the direct bearing stress or the diagonal force divided by an arbitrary area. All three active node geometries must be checked because their orientations and capacities differ.

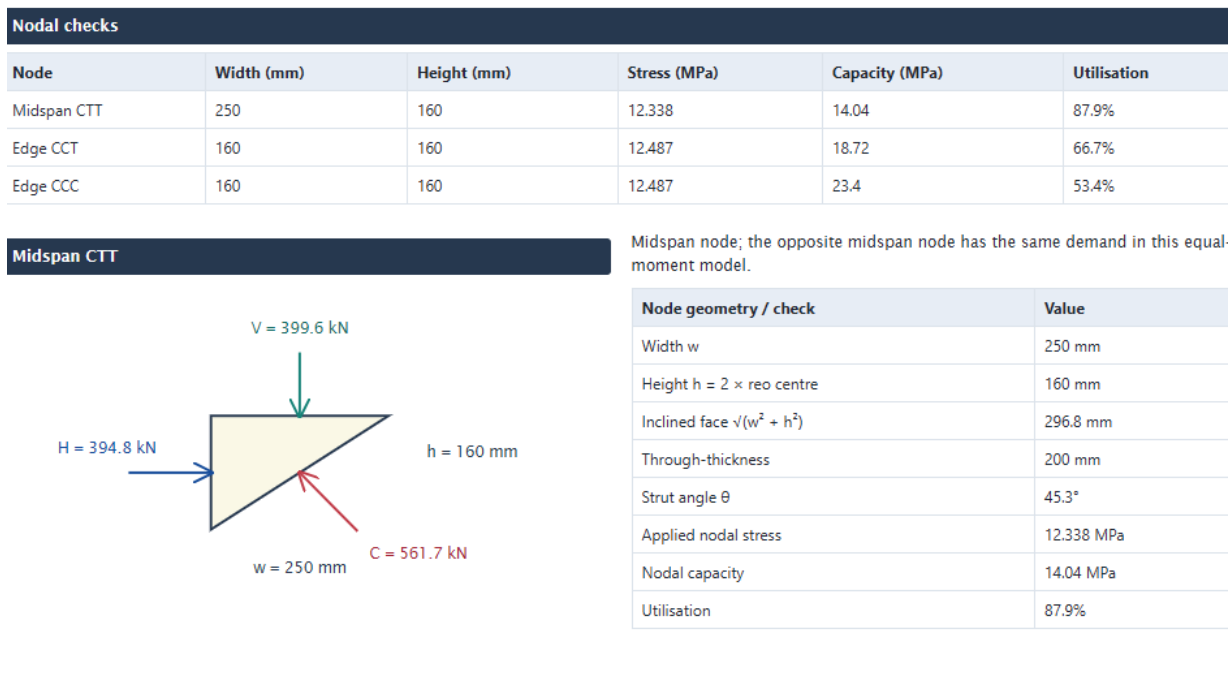


Figure 7.4 – Nodal Check of Spandrels

7.9. Nodal resistance and utilisation

The adopted unconfined nodal resistance combines $\phi_n = 0.65$, a concrete factor of 0.90, and a node type efficiency β_n .

$$\sigma_{\text{capacity}} = 0.65 \times 0.90 \times \beta_n \times f'_c$$

Active node	Width	Height	β_n	Capacity for f'_c of 40 MPa
Midspan CTT	w _m	2c	0.60	14.04 MPa
Edge CCT	w _e	2c	0.80	18.72 MPa
Edge CCC	w _e	2c	1.00	23.40 MPa

$$U_{\text{node}} = 100\sigma_{\text{node}}/\sigma_{\text{capacity}} \quad \text{in percent}$$

The reported spandrel utilisation is the maximum of the three percentages. PASS means that these nodal stress comparisons do not exceed 100 percent. The calculated tie areas are demands for subsequent reinforcement detailing, not separate provided reinforcement utilisation checks.

7.10. Completing the strut and tie design

A complete strut and tie design requires a viable mechanism in which every strut, tie and node can transfer its force. The current spandrel status checks the active nodal stresses and calculates the tie areas. It does not calculate a complete diagonal strut resistance or bursting reinforcement design.

The designer must verify strut effective width and strength, transverse splitting or bursting reinforcement, tie anchorage, confinement, available anchorage into the adjoining piers and the compatibility of the longitudinal reinforcement with the shear mechanism. Additional checks may be needed for reversal and for transfer of the coupling beam actions into the wall boundaries.

Intermediate section actions, serviceability, out of plane actions, torsion and axial interaction are outside the current spandrel calculation. A passing node should not be used to waive those assessments or to approve a reinforcement arrangement that cannot be placed and developed.

8. Design Results and Calculation Memos

8.1. Pier section results

The pier results identify the selected bar diameter, bars per face, total steel area, longitudinal percentage and spacing. An expression such as 11 N16 EF means eleven 16 mm vertical bars on each face, twenty two bars in total. The spacing is measured along the section length between bar centres and is calculated from the available length between the two reinforcement centre offsets.

The section action table retains the original P, M22 and M33 and gives the adopted design moments after minimum eccentricity and any selected magnification. It reports the out of plane magnifier and stability information, the available moment capacities at that row axial force, and the governing utilisation. Spectrum rows are identified as component peak envelopes.

The weak and strong axis interaction diagrams illustrate the governing section row, rather than every row that was checked. The biaxial diagram is shown only where the governing row actually used that check. A designer should consult the complete action table when a nongoverning combination is of particular interest.

8.2. Pier shear and horizontal reinforcement results

The shear report gives H/L, the concrete and reinforcement contributions, nominal web crushing capacity, the adopted factors, and the final design capacity. The required, provided and effective ratios have different meanings: required is the horizontal demand envelope; provided comes from the selected horizontal schedule; effective is the ratio used in V_{us} after any squat wall lesser direction restriction.

The horizontal schedule uses EF in the same way as the vertical schedule. For example, N12 at 300 mm EF supplies one 12 mm horizontal bar at each 300 mm spacing on each face. Its total area per metre includes both faces. Failure to preserve this distinction can double or halve the intended reinforcement in a drawing.

The shear action table records signed original shear, spectrum contribution where applicable, amplified shear, design magnitude and utilisation for every checked end. A nominal factor beside a row should be read with its demand method: Ω acts only on the spectrum part in the component method, while the legacy multiplier acts on the complete row shear.

8.3. Boundary results

Boundary output includes the overall model height, adopted height branch, two edge zone lengths, restraint categories, maximum compression and separate governing rows for length and category. The individual stress table provides the signed action based edge stresses and calculated lengths at each checked section.

Boundary states include REQUIRED, NOT REQUIRED BY SCREENING, REVIEW REQUIRED and PARTIAL. These describe the scope of this screening. They are independent of the section and shear PASS or FAIL. Where whole wall core confinement is flagged, an edge length of zero does not remove that requirement.

8.4. Spandrel results

The spandrel memo reports geometry and materials, the selected combinations, original and spectrum shear components, amplified shears, and the independently governing shear and flexural combinations. Its equilibrium summary distinguishes the equal equivalent shear model moments from the original moments used for longitudinal ties.

The nodal cards show width, height, inclined face geometry, force directions, applied stress, resistance and utilisation. The midspan CTT, edge CCT and edge CCC checks all contribute to the reported nodal status. Reinforcement output consists of the distributed vertical tie demand and the left and right longitudinal tie area envelopes.

Because the spandrel routine does not select a complete provided bar schedule, a PASS badge must be read together with these calculated reinforcement demands and the scope of the model. A node stress pass does not establish that an adopted tie detail supplies or develops the required areas.

8.5. Status and coverage

Result	Interpretation	Required engineering response
PASS for pier	Available section and shear checks pass with complete reported coverage	Complete boundary detailing and other excluded assessments
FAIL	One or more implemented strength stability or fit checks fail	Revise demand geometry or reinforcement settings and redesign
PARTIAL	Some selected action coverage is unavailable	Resolve missing cases or ends and rerun the complete set
ERROR	Inputs geometry or analysis data cannot support calculation	Correct the stated cause before interpreting a design
PASS for spandrel	All three reported node stress checks pass	Supply and develop calculated ties and complete the mechanism checks

The numeric utilisation is retained at full calculation precision; displayed rounding does not change the decision. A displayed value close to 100 percent should be assessed using the recorded status and more precise values, rather than a manually rounded comparison.

8.6. Model overlays and exports

Pier overlays display the longitudinal reinforcement and a second line for horizontal reinforcement. Reinforcement colours assist comparison, while failed members are identified explicitly. Hover information and the member memo provide the governing check. The overlay represents a valid saved calculation result, not an additional independent analysis.

Calculation memos can be printed or saved to PDF through the application. CSV exports preserve the numerical checks, combinations and coverage information for further review. The engineer should retain the model and selected combination definitions with the issued results so that the design basis can be reproduced.

8.7. Result invalidation

A result is valid only for the geometry, materials, design settings, combination definitions and analysis results used to calculate it. Relevant changes invalidate the displayed result, and responses that arrive after an input change are not committed as current designs. Changes to overall model height also invalidate the pier boundary classification.

Saved results are restored only where their signatures and design method versions remain compatible. The current pier response includes boundary screening in version 5, and the spandrel response uses version 2. Version checks prevent earlier shear or boundary methods from being silently presented as current calculations. A stale result must be rerun rather than reinterpreted under the new theory.

9. Recommended Design Workflow

9.1. Establish the structural model

Define the actual wall geometry, openings, thicknesses, materials and connections to slabs, beams and foundations. Select a mesh appropriate to the force gradients and verify connectivity across the intended load path. Review stiffness assumptions, supports and the analysis treatment of seismic actions before local design.

Assign pier and spandrel labels so that their complete extents represent the intended rectangular design members. Keep discontinuous pieces separate and split members where an opening, taper or section change prevents the rectangular idealisation. Confirm that the spandrel span corresponds to the intended clear distance between pier faces.

9.2. Verify actions before reinforcement design

Run the required analysis cases and check reactions, deformations and global equilibrium. In Results, inspect the member end force tables and local axes for representative piers and spandrels. Confirm compression signs under gravity and the bending and shear senses under lateral load reversal.

Check that all selected combinations have valid source cases and complete recovered wall actions. For spectrum cases, verify the modal and directional settings, scaling and the member integration process. A smooth stress contour is not evidence that complete design end actions are available.

9.3. Set pier design preferences

Confirm longitudinal and horizontal yield strengths, reinforcement centre and minimum bar diameters. Set exposure, horizontal restraint and crack control level consistently with the intended detail. Select the shear reduction factor and document the fitment qualification if the conditional 0.75 option is used.

For every pier, review H used for shear. If out of plane buckling is selected, supply a defensible effective length and sustained load ratio. Confirm the model height proxy used for boundary screening and the nominated ductility. Where a spectrum component envelope is used, set Ω for the required design basis; where the retained static seismic path is used, confirm the explicit earthquake combination selections and μ/S_p inputs.

9.4. Run and review the unified pier design

Select the intended ULS combinations and run Design piers. The routine seeks one longitudinal arrangement that satisfies the selected section and shear action sets. Inspect overall status, available action coverage, governing section and shear rows, and both reinforcement schedules.

Review the minimum eccentricity moments and any magnification. For squat walls, compare the vertical, horizontal and effective shear ratios. If a wall fails in crushing or stability, changing only the reinforcement minimum is unlikely to resolve the underlying mechanism; revise the relevant section, restraint or demand assumptions.

Review the independent boundary output even where the strength result passes. Adopt appropriate edge extents and categories, then design the actual ties or U bars. Complete whole core, axial force and tension zone confinement assessments where relevant.

9.5. Set and run spandrel design

Confirm the model derived span, depth, thickness and concrete strength. Enter edge and midspan node widths and reinforcement centres that correspond to feasible nodal and tie details. Confirm f_y , f_{ys} and the nominated spectrum shear factor, then design the selected spandrels for the intended combinations.

Inspect shear and longitudinal governing combinations separately. Check all three nodal stress cards and the calculated vertical and longitudinal tie steel. Compare the original moment tie demand with the equal moment shear mechanism and provide a reinforcement layout capable of carrying the complete force system under the required action reversals.

9.6. Complete details and record the design

Develop the reinforcement drawings with cover, layer order, practical spacing, laps, anchorage and confinement. Check force transfer into adjoining members and foundations. Complete serviceability and other actions outside the current routines, including out of plane shear and torsion where relevant.

Resolve every missing action or partial result before issuing a complete design record. Rerun calculations after changes to the model, material, settings or load combinations. Retain the calculation memos, action tables and model revision together so that the reported reinforcement can be traced to its actual demand envelope.

10. Assumptions and Application Limits

10.1. Geometry and reinforcement idealisation

The present design routines cover planar rectangular piers and spandrels with uniform thickness and concrete strength. They do not design nonrectangular cores, flanged composite wall sections, taper or reinforcement around internal openings. Broader member force reporting does not remove these section design restrictions.

Pier reinforcement is symmetric on two faces with uniform spacing along the length. Concentrated end cages, mixed diameters, unequal reinforcement faces and nonuniform sections need a capacity check that represents the adopted arrangement. The 4 percent longitudinal upper bound is the limit of this search, not a universal permission to detail that ratio in any wall.

10.2. Analysis and action limitations

Only the recovered end stations are designed. A straight diagram between them does not establish interior section capacity or capture every local force concentration. The analysis model must adequately represent load introduction, wall to floor transfer, coupling action, openings and foundations.

Spectrum rows use selected combination component peaks. They do not demonstrate a concurrent stress state, and the program does not create all sign permutations automatically. The designer remains responsible for the action convention and the selected combinations needed to satisfy the project design basis.

10.3. Seismic and confinement scope

The nominated Ω , ductility and performance factors are engineering inputs. Their presence in a memo does not establish a complete capacity design hierarchy or the suitability of a wall or coupling beam ductility class. The legacy and spectrum component shear methods must be distinguished when reviewing the calculations.

Boundary screening calculates elastic edge extents and categories at Top and Bottom. It does not proportion fitments, verify their development, recognise every existing restraint detail or assess intermediate heights. Separate axial force confinement requirements, critical tension zones and connection detailing remain outside its status.

10.4. Stability and serviceability

The optional pier out of plane magnifier is a local member calculation. It does not establish whole building stability, replace a required second order analysis, or assess every possible out of plane wall instability mechanism. Leaving the option unselected requires the engineer to have assessed the wall restraint and the relevant design basis.

Neither wall routine performs a complete crack width, long term deflection, vibration, shrinkage restraint or fire resistance assessment. The horizontal minimum ratios address the implemented reinforcement provisions; actual serviceability may require additional reinforcement or a different structural detail.

10.5. Reinforcement and force transfer

Cover and reinforcement centres are different quantities. Centres must allow for cover, bar radius and any outside transverse reinforcement. The program fit checks do not establish all placing tolerances, aggregate clearance or the geometry of laps and anchorage zones.

For spandrels, nodal stress acceptance and calculated tie steel must be supplemented by strut strength, splitting and bursting reinforcement, anchorage and a complete compatible load path. For piers, a passing

distributed steel arrangement must be supplemented by boundary and connection details. The final design must be checked using the reinforcement that will actually be constructed.

10.6. Numerical implementation

The interaction curves are sampled polygonal envelopes, with inherited strength reduction and high compression construction. Pier moment capacity is interpolated at the applied axial level. Steel area rounding in the capacity routine is separate from the exact area used in the schedule and ratio report.

Floating point results are used without deliberately rounding demand downward. Display rounding is for readability. Independent checks should use consistent units, the same action signs and the implemented curve and reinforcement idealisations before a discrepancy is attributed to a physical design difference.

Appendix A Worked Calculation Examples

A.1 Signed spectrum shear amplification

Consider a selected combination with a static contribution of +100 kN and a factored spectrum contribution of -180 kN at one end. The reported original combination shear is $V = -80$ kN. Let $\Omega = 2.0$.

$$V_{\text{amplified}} = -80 + (2 - 1)(-180) = -260 \text{ kN}$$

$$V_{\text{design,end}} = \max(80, 260) = 260 \text{ kN}$$

Multiplying the original magnitude by two would give 160 kN and would miss the signed component method. The static contribution remains +100 kN; the spectrum term becomes -360 kN.

For a second example, take a static contribution of +300 kN and spectrum contribution of -200 kN, again with $\Omega = 2$. The original shear is +100 kN and the amplified shear is -100 kN. The design demand is 100 kN. The calculation does not sum the two magnitudes or replace the signed spectrum contribution by an unsigned value.

These examples demonstrate the end equation only. The complete design must still envelope all required ends and selected combinations.

A.2 Pier shear with a supplied arrangement

Take $L = 3000$ mm, $t = 200$ mm, $H = 6000$ mm, $f'_c = 40$ MPa, $f_y = f_{ys} = 500$ MPa and $\phi_v = 0.70$. Assume a supplied longitudinal arrangement of 11 N16 EF and horizontal reinforcement N12 at 300 mm EF. Use a design shear of 500 kN. This example checks the stated arrangement for shear; it does not claim that it is the minimum arrangement selected by a full section and shear search.

$$H/L = 2.0 \quad A_v = 0.8 \times 3000 \times 200 = 480000 \text{ mm}^2$$

$$k_v = \max[0.17, \min(0.24, 0.15)] = 0.17$$

$$V_{uc} = 0.17\sqrt{40} \times 480000/1000 = 516.08 \text{ kN}$$

The total vertical area is $22\pi 16^2/4 = 4423.36 \text{ mm}^2$, giving $\rho_v = 0.007372$. Its spacing is $(3000 - 108)/10 = 289.2$ mm for $c = 54$ mm. The horizontal ratio is:

$$\rho_h = 2(\pi 12^2/4)/(300 \times 200) = 0.003770$$

Because $H/L > 1$, ρ_{eff} is the horizontal ratio. For restrained A1 minor crack control, the base minimum is 0.0025 and the supplied horizontal ratio exceeds it. The centre spacing 300 mm is below 350 mm and exceeds the 48 mm minimum centre spacing for N12.

$$V_{us} = 0.00376991 \times 500 \times 480000/1000 = 904.78 \text{ kN}$$

$$0.70V_{u,\max} = 0.70 \times 0.2 \times 40 \times 480000/1000 = 2688.00 \text{ kN}$$

$$V_{\text{capacity}} = \min[0.70(516.08 + 904.78), 2688.00] = 994.60 \text{ kN}$$

$$U_{\text{shear}} = 500/994.60 = 0.503 \quad U_{\text{crushing}} = 500/2688 = 0.186$$

The strength ratio from shear alone is zero because V_{design}/ϕ_v is less than V_{uc} . Minimum steel therefore controls the ratio demand in this example. The selected supplied horizontal schedule is more than the minimum and is intentionally retained for this independent arithmetic check.

If H were changed to 2400 mm while the same section and schedules were retained, H/L would become 0.8. The coefficient would be 0.492 and ρ_{eff} would become $\min(\rho_v, \rho_h)$. If a lower longitudinal ratio than the

horizontal ratio were supplied, it would limit the steel shear contribution. This is why the actual program checks shear during the longitudinal search.

A.3 Boundary zone under signed bending

Take the same 3000 by 200 mm rectangle with $f'_c = 40$ MPa, analysis $P = -1200$ kN, $M_{33} = +1800$ kN m and $M_{22} = 0$. Compression from axial load is 2.0 MPa and the in plane bending edge stress magnitude is 6.0 MPa.

$$\sigma_1 = 2 - 6 = -4 \text{ MPa} \quad \sigma_2 = 2 + 6 = 8 \text{ MPa}$$

$$q = 0.15 \times 40 = 6 \text{ MPa}$$

Only Edge 2 exceeds the threshold. Its crossing length is:

$$l_2 = 3000(8 - 6)/[8 - (-4)] = 500 \text{ mm}$$

For a limited ductile building at the low rise height branch, the deemed extent is $\max(500, 1.5 \times 200) = 500$ mm and the category is the U bar or closed tie arrangement of 14.6.2.2. At the taller branch, 8 MPa is exactly $0.20f'_c$ and does not exceed the implemented strict enhanced trigger. The category remains closed fitments to 14.6.2.3 and 10.7.4 for that row. A value greater than 8 MPa would activate the enhanced category.

Reversing M_{33} gives $\sigma_1 = 8$ MPa and $\sigma_2 = -4$ MPa, moving the same 500 mm requirement to Edge 1. If both reversal combinations are selected, both edge envelopes are required. Adding a nonzero M_{22} increases the governing face compression at both edges and requires recalculating the crossings.

A.4 Minimum eccentricity and biaxial evaluation

For a 3000 by 200 mm section under $N^* = 1200$ kN compression, the minimum eccentricities are 10 mm out of plane and 150 mm in plane. With analysis $M_{22} = 0$ and $M_{33} = 100$ kN m, and buckling unselected, the adopted moments become:

$$M_{22,\text{design}} = \max(0, 1200 \times 0.010) = 12 \text{ kN m}$$

$$M_{33,\text{design}} = \max(100, 1200 \times 0.150) = 180 \text{ kN m}$$

Both adopted moments are nonzero, so the biaxial check is applicable despite the zero analysis M_{22} . To illustrate the final arithmetic, suppose a trial arrangement has reduced envelope capacities at this axial force of $M_{22,b} = 40$ kN m and $M_{33,b} = 900$ kN m, with nominal $N_{uo} = 20000$ kN. These assumed capacities are explanatory inputs, not output claimed for the example section.

$$\alpha = \max[1, \min(2, 0.7 + 1.7 \times 1200/(0.65 \times 20000))] = 1.0$$

$$U_{\text{biaxial}} = (12/40)^1 + (180/900)^1 = 0.50$$

The actual design must calculate the curve capacities for its selected arrangement and also check the uniaxial, axial and stability comparisons. If out of plane magnification produced $\delta_{22} = 2.0$, the first numerator would become 24 kN m and this illustrative biaxial value would increase to 0.80.

A.5 Spandrel force and tie calculation

Take $f'_c = 40$ MPa, $f_y = f_{ys} = 500$ MPa, $l = 2000$ mm, $D = 1000$ mm, $t = 200$ mm, $w_e = 160$ mm, $w_m = 250$ mm and $c = 80$ mm. Suppose the selected shear envelope after the required spectrum treatment is 300 kN. Let the governing original end moment magnitudes be 250 kN m left and 200 kN m right.

$$z = 1000 - 160 = 840 \text{ mm} \quad L_w = 2000 + 160 = 2160 \text{ mm}$$

$$M_{eq} = 300 \times 2160/2000 = 324.00 \text{ kN m}$$

$$\theta = \text{atan}(1680/2160) = 37.875 \text{ degrees}$$

$$C = 300/\sin\theta = 488.65 \text{ kN} \quad T_h = 324 \times 1000/840 = 385.71 \text{ kN}$$

$$A_v = 300000/(0.85 \times 500) = 705.88 \text{ mm}^2$$

$$A_v/s = 705.88/2000 = 0.35294 \text{ mm}^2/\text{mm} = 352.94 \text{ mm}^2/\text{m}$$

The original moment longitudinal tie areas are independent of the 324 kN m equivalent moment:

$$A_{s,\text{left}} = 250 \times 10^6/(0.85 \times 500 \times 840) = 700.28 \text{ mm}^2$$

$$A_{s,\text{right}} = 200 \times 10^6/(0.85 \times 500 \times 840) = 560.22 \text{ mm}^2$$

The 700.28 mm² envelope is the largest original moment tie demand in these assumed inputs. The horizontal force in the shear mechanism corresponds to a different tie demand and must be considered when completing a compatible reinforcement layout.

A.6 Spandrel nodal stress check

Use the spandrel data of A.5. Every node has $h = 160 \text{ mm}$. The midspan node width is 250 mm and the edge widths are 160 mm. The table gives the intermediate values from the equations in Sections 7.7 to 7.9, retaining unrounded calculation values internally.

Quantity	Midspan CTT	Edge CCT	Edge CCC
Width mm	250	160	160
Inclination ψ degrees	19.506	7.125	7.125
Direct bearing MPa	6.000	9.375	9.375
Direct side stress MPa	12.054	12.054	12.054
Inclined face stress MPa	8.733	10.882	10.882
Transformed normal σ_n MPa	7.759	10.714	10.714
Transformed shear τ MPa	2.749	1.339	1.339
Controlling node stress MPa	12.054	12.054	12.054
Design resistance MPa	14.040	18.720	23.400
Utilisation percent	85.852	64.389	51.511

The governing reported node is Midspan CTT at 85.852 percent. These particular dimensions give the same controlling stress at all three nodes, but that is not an assumed equality in the calculation. Different widths or strut angles can change the governing applied stress as well as the governing resistance.

The example passes the three nodal comparisons. It still requires the vertical and longitudinal tie reinforcement, a compatible complete mechanism, diagonal strut and bursting checks, and anchorage and confinement details. It is not a construction detail or an overall coupling beam compliance certificate.

Appendix B Symbols and Units

Symbol	Meaning	Working units
P	Reported member axial force positive in tension	kN
N*	Section axial demand positive in compression equal to -P	kN
M22 and M33	Pier out of plane and in plane moments	kN m
M3	Spandrel analysis bending moment	kN m
V2	Raw integrated in plane design shear	kN
V ^E	Signed factored spectrum shear contribution	kN
Ω	Nominated amplification of spectrum shear contribution	Dimensionless
L and t	Pier section length and thickness	mm
H	Engineer nominated wall height for shear aspect ratio	mm
l _e	Optional out of plane buckling effective length	mm
c	Face to reinforcement centre dimension	mm
f'c	Concrete compressive strength	MPa
f _y and f _{ys}	Longitudinal and transverse steel yield strengths	MPa
p _v and p _h	Total vertical and horizontal steel ratios including both faces	Decimal ratio
φ	Mechanism dependent capacity reduction factor	Dimensionless
μ and S _p	Ductility and performance factors	Dimensionless
l and D	Spandrel clear span and depth	mm
w _e and w _m	Edge and midspan node widths	mm
L _w and z	Spandrel working span and truss depth	mm
T _v and T _h	Vertical and horizontal tie forces in shear model	kN
C and θ	Diagonal strut force and angle to horizontal	kN and degrees
A _{v/s}	Required total vertical tie steel per unit clear span	mm ² /mm
A _{s,end}	Longitudinal tie area from original analysis end moment	mm ²
EF	Reinforcement schedule applies at each face	Not a multiplier on total areas

Stress in MPa is N/mm². A force in kN is multiplied by 1000 when used with mm², and a moment in kN m is multiplied by 1000000 when used with mm dimensions. Reinforcement percentages are one hundred times decimal ratios. Pier utilisation is stored as a ratio; spandrel nodal utilisation is returned as a percentage.

Appendix C Design Basis and Calculation Traceability

C.1 Standards and clause map

AS 3600:2018 Concrete structures, incorporating Amendment 1 of November 2018 and Amendment 2 of May 2021, is the stated basis for the implemented design. Standards Australia lists the 2018 standard and its amendment records in its official catalogue. The catalogue entry establishes the publication identity; the equations documented here are traced to the project implementation.

Standards Australia catalogue: <https://store.standards.org.au/search?query=AS3600-2018>

Subject	Principal reference used by the implementation
Capacity reduction factors	Table 2.2.4 and applicable mechanism provisions
Steel tension ties	Section 7 and Clause 7.3.2
Unconfined nodal regions	Clause 7.4.2
Conditional shear factor detailing	Clause 8.2.1.7
Rectangular section interaction and restraint	Section 10 including 10.7.4 as applicable
Pier in plane shear	Clauses 11.6.1 to 11.6.4
Wall minimum steel spacing and crack control	Section 11.7
Nonductile boundary assessment	Clause 11.7.4
Boundary restraint and high strength core confinement	Clauses 14.6.2 and 14.6.3 with 14.5.4
Retained capacity based seismic shear path	Equation 14.6.6
Critical tension reinforcement zones	Clause 14.6.7 as a separate designer check

This clause map identifies where the design concepts are addressed. It does not imply that every requirement within a referenced section is automatically checked. The implemented assumptions and remaining assessments are stated in the corresponding chapters.

Appendix D Design Settings and Defaults

The initial values below apply to a new model before its design settings are changed. Saved model settings take precedence. Defaults are starting values for engineering review; they do not establish the appropriate design assumptions for a particular structure.

D.1 Model wide materials and pier preferences

Setting	Units	Initial value	Scope	Effect on calculation
Longitudinal yield strength f_y	MPa	500	Global	Material strength for pier section resistance and spandrel longitudinal ties.
Transverse yield strength f_{ys}	MPa	500	Global	Material strength for pier horizontal reinforcement and spandrel vertical ties.
Pier reinforcement centre	mm	54	Global piers	Face to vertical bar centre; affects section capacities, spacing and cage fit.
Minimum vertical diameter	mm	12	Global piers	Lower bound on standard diameters considered in the longitudinal search.
Minimum horizontal diameter	mm	12	Global piers	Lower bound on standard diameters considered for bars on both faces.
Out of plane buckling	—	Not selected	Global piers	When selected, enables the out of plane magnifier for compressive rows. In plane magnification remains 1.
Exposure class	—	A1	Global piers	Together with restraint and crack control, sets the horizontal minimum ratio.
Crack control level	—	Minor	Global piers	Minor, Moderate or Strong; affects restrained horizontal minimum steel.
Horizontal movement restrained	—	Yes	Global piers	Selects whether the restrained shrinkage and temperature minimum applies.
General shear factor ϕ_v	—	0.70	Global piers	Alternative 0.75 assumes qualifying fitment detailing; web crushing retains 0.70.
Spectrum shear factor Ω	—	1.0	Global piers	Amplifies only the signed spectrum contribution. Ω must be at least 1.
Ductility μ	—	2.0	Global piers	Used in boundary classification and the retained static seismic shear path. Supported range is 1 to 3.
Performance factor S_p	—	0.77	Global piers	Used in the μ/S_p limit of the retained static seismic shear path.
Earthquake combinations	—	None selected	Global piers	Explicit selection for the retained static seismic path. Spectrum contributions are recognised from analysis metadata.

Global settings apply to all piers of the model. The pier wall procedure limits f_y and f_{ys} to 500 MPa. The required design combinations must be selected for the design run; their coverage is reported with the results.

D.2 Properties assigned to each pier

These properties must be consistent across all source shells carrying the same pier label. Their initial geometric values are not a substitute for an assessment of the actual wall system and its restraint.

Setting	Units	Initial value	Scope	Effect on calculation
Wall height H for shear	mm	Labelled height	Per pier	Defines H/L in the wall shear equations. Confirm the appropriate structural wall height.
Out of plane effective length	mm	Labelled height	Per pier	Used only when out of plane buckling is selected; affects critical load and magnification.
Sustained load ratio β_d	—	1.0	Per pier	Used in the out of plane critical load calculation. Permitted range is 0 to 1.

D.3 Properties obtained from the model

Setting	Units	Initial value	Scope	Effect on calculation
Concrete strength f'_c	MPa	Assigned material	Derived	One common concrete strength is required through each labelled member.
Wall thickness t	mm	Source shells	Derived	One common thickness is required through each labelled member.
Pier section length L	mm	Horizontal extent	Derived	Length of the complete straight rectangular pier section.
Labelled pier height	mm	Vertical extent	Derived	Provides the initial shear height and optional effective length.
Building height	mm	Overall vertical extent	Derived	All frame and shell elevations contribute. Above 12 m selects the adopted taller building boundary branch.
Spandrel clear span l	mm	Horizontal extent	Derived	The full labelled extent is used as the clear span; labels must represent the intended beam faces.
Spandrel overall depth D	mm	Vertical extent	Derived	Together with reinforcement centre determines the truss depth.

Labelled height, shear height H, buckling effective length and overall building height have distinct roles. Altering one does not justify substituting it for another. Model dimensions are converted from metres to millimetres for design.

D.4 Model wide spandrel preferences

Setting	Units	Initial value	Scope	Effect on calculation
Edge node width w_e	mm	160	Global spandrels	Sets the edge nodal geometry and the addition to clear span in $L_w = l + w_e$.
Midspan node width w_m	mm	250	Global spandrels	Sets the active midspan CTT nodal width.
Reinforcement centre c	mm	80	Global spandrels	Equal top and bottom tie centres; gives truss depth $z = D - 2c$.
Spectrum shear factor Ω	—	1.0	Global spandrels	Amplifies only signed spectrum shear contributions; original end moments remain unchanged.

Spandrels use the global f_y and f_{ys} material strengths listed in F.1. Pier and spandrel reinforcement centres and spectrum factors are separate preferences.

D.5 Geometry checks and adopted calculation rules

The spandrel geometry must satisfy $2c < D$, $w_e < l/2 - 100$ mm, and $w_m \leq l$. Reinforcement centre is measured to the bar or tie centre and is not concrete cover. The selected dimensions must represent a feasible detail.

Some values are fixed rules of the current calculation rather than adjustable preferences. Pier longitudinal reinforcement is searched between 0.25% and 4% total steel, using equal rows on both faces. Maximum bar spacing is $\min(2.5t, 350$ mm); the clear spacing criterion is 3 bar diameters. Horizontal spacings are selected in 25 mm increments.

Spandrel steel ties use a capacity reduction factor of 0.85. Unconfined nodal checks use 0.65 with the 0.90 concrete factor and node efficiency factors 0.60, 0.80 and 1.00 for CTT, CCT and CCC respectively. Pier section reduction factors follow the interaction calculation; they are not a single global ϕ setting.

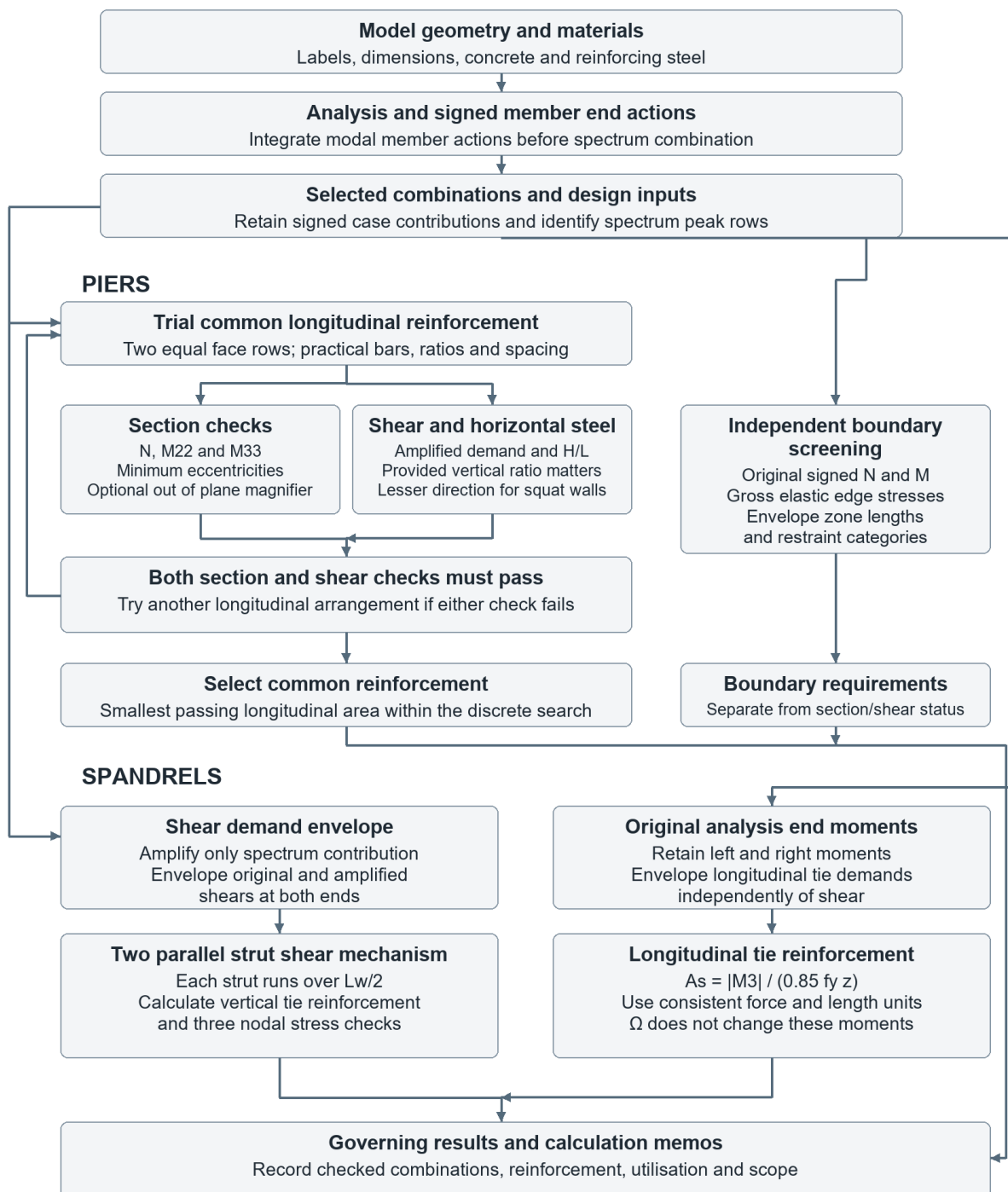
D.6 Effect of changing settings

Changes to relevant settings invalidate the previous design results. Rerun the affected members and review combination coverage, reinforcement fit, governing checks and boundary requirements. A change in geometry or reinforcement settings does not itself perform a new structural analysis.

Design settings establish calculation inputs. They do not select or verify the complete anchorage, lap and confinement arrangement. The calculation memo records the adopted assumptions and the scope of the reported checks.

Appendix E Shear Wall Design Calculation Map

The calculation paths below connect the selected analysis combinations to the reported pier and spandrel design results. Spectrum section rows retain their component peak envelope qualification.



Pier section and shear status is separate from boundary detailing.

Spandrel PASS refers to nodal checks; calculated ties require detailing.

Incomplete action coverage is reported and cannot establish a complete PASS.

Appendix F Analysis Validation

F.1 Introduction

The repaired automatic wall mesher generates the coupled shear wall directly from 39 source panels into 858 rectangular shells. All shell polygons and 1,040 nodes match the ETABS mesh after coordinate alignment. The current horizontal-load roof displacement is 40.3465 mm against 41.6910 mm in ETABS, a 3.22% difference. Overall equilibrium agrees closely; local member differences remain and are evaluated using absolute force differences.

F.2 Scope and verification basis

This report compares a thirteen-storey concrete frame and coupled wall using self-weight and a separate horizontal load case. The evidence combines independent static analyses, member-end force recovery, matched frame stations, joint translations and the supplied graphical comparisons. It concerns the stated linear elastic model and does not establish general mesh independence or concrete member resistance.

Evidence	Coverage	Finding
Piers and spandrels	26 piers and 13 spandrels; six components at both ends	Automatic mesh: all four end-edge segments at every spandrel connect to piers
Frame actions	13 beams and 13 columns; all ETABS output stations	Residual axial, shear and moment differences remain
Displacements	1,040 corresponding ETABS joints	Live roof Ux 3.22% lower in FEAkalc in the current automatic-mesh run
Equilibrium	Six global base resultants in each case	Gravity total 1,527.5 kN and horizontal total 1,300 kN closely reproduced
Graphical evidence	16 response screenshots and two loading screenshots	Original response screenshots retained as historical visual evidence

Numerical tables and the mesh figure use the 25 October rerun of file 1 with automatic meshing and a fresh ETABS export. The supplied response screenshots and file 3 workbook remain historical records; their labels are not the current rerun values.

F.3 Problem definition and loading

The model is a planar frame coupled to two wall piers by thirteen spandrels. Storeys are 3 m apart. FEAkalc has five authored fixed supports; eight additional wall-base nodes inherit Ux, Uy and Uz. Rotations remain fixed only where explicitly assigned. ETABS fixes all six freedoms at its thirteen base nodes. Dimensions below use ETABS coordinates.

Input	Adopted value
Height and framing	13 storeys at 3.0 m; total height 39.0 m; frame bay 4.0 m

Input	Adopted value
Wall geometry	Left pier width 3.0 m; right pier width 2.0 m; opening width 1.5 m
Wall and spandrel sections	Wall thickness 0.20 m; spandrel depth 1.0 m; span 1.5 m
Frame section	200 × 1,000 mm concrete rectangle; $A = 0.20 \text{ m}^2$
Elastic properties	$E = 32,800 \text{ MPa}$; Poisson ratio 0.20; concrete unit weight 25 kN/m^3
Dead case	Own weight only, acting vertically
Live case	100 kN global +X at the frame column on each floor; total 1,300 kN

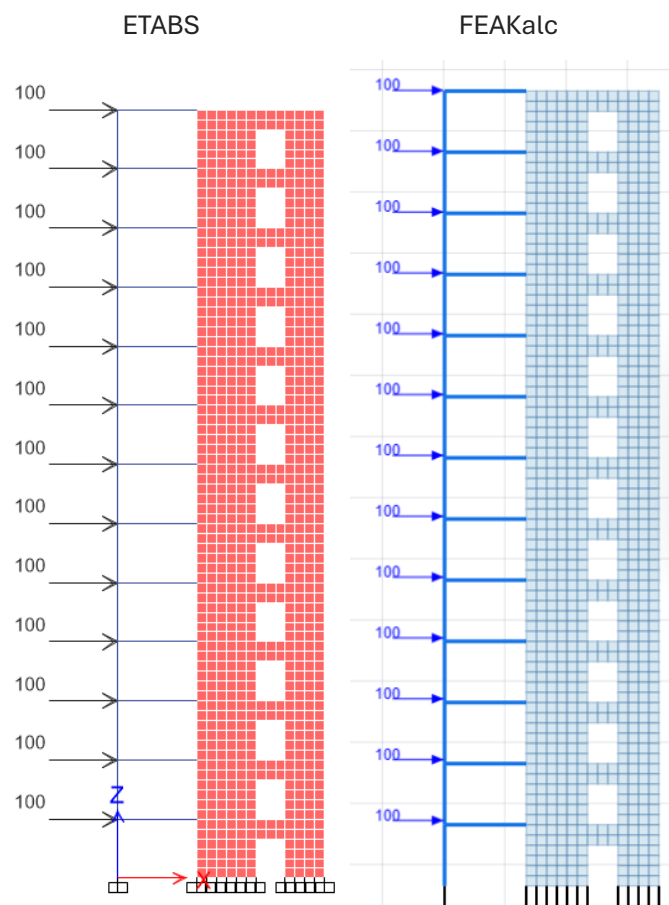


Figure 1. Supplied loading views. Each arrow represents 100 kN horizontal loading in the Live case. Dead is a separate self-weight case; Live is not a scaled Dead response.

F.4 Analysis meshes and connectivity

FEAkalc now meshes the original 39 wall and spandrel polygons automatically at 0.5 m. The rectangular-wall engine generates 858 quadrilaterals with no triangles. ETABS retains 858 manually divided quadrilaterals. All 858 element polygons and 1,040 nodes match after removing the FEAkalc coordinate offset; this is now a matched shell-layout comparison.

Measure	ETABS	FEAKalc automatic mesh
Source wall and spandrel panels	858	39 undivided polygons
Analysis shell elements	858 quads	858 quads; no triangles
Total model nodes	1,040	1,040
Total wall area	214.500000 m ²	214.500000 m ²
Coordinate relation	Reference coordinates	ETABS + (0.05, 0, 1.95) m

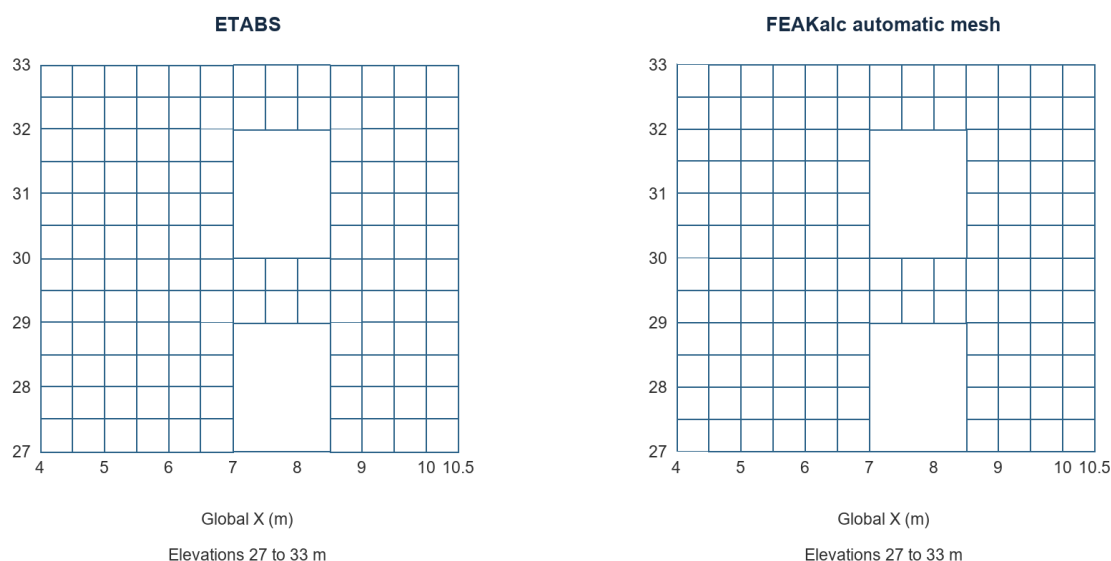


Figure 2. Current exported analysis meshes around B10 and B11 at elevations 27 to 33 m. Element edges are drawn directly from both result meshes. FEAKalc coordinates are translated into the ETABS system; the rectangular layouts coincide.

Automatic meshing now resolves the earlier B10/B11 connection anomaly without manual panel division. Every spandrel B1 to B13 has four end-edge segments shared with adjacent piers, including all B10 and B11 segments. The five authored supports plus eight inherited base nodes are retained in assembly.

Both models have 6,240 total DOF before restraints. FEAKalc has 6,186 free DOF after its 54 restraints. Matching all thirteen ETABS base restraints in a controlled copy changes Live roof Ux by only 0.0051 mm, so that rotational-restraint difference does not explain the remaining displacement discrepancy.

F.5 Numerical results and interpretation

Translations below refer to the corresponding roof point at ETABS (0, 0, 39) m. FEAkalc deformation output is converted to physical sign and millimetres. A signed difference is FEAkalc minus ETABS; the percentage denominator is the absolute ETABS value.

Case	Component	FEAkalc mm	ETABS mm	Difference mm	Difference
Dead	Ux	-1.0298	-1.0320	0.0022	+0.21%
Dead	Uz	-1.0286	-1.0280	-0.0006	-0.06%
Live	Ux	40.3465	41.6910	-1.3445	-3.22%
Live	Uz	4.3899	4.3110	0.0789	+1.83%

Base resultants are evaluated about the ETABS origin, including the coordinate transformation of reaction moments. Values are shown at export precision; very small differences should be interpreted with that rounding in mind.

Case	Resultant	FEAkalc	ETABS	Difference
Dead	FZ kN	1,527.5001	1,527.5000	0.0001
Dead	MY kN.m	-8,198.1255	-8,198.1250	-0.0005
Live	FX kN	-1,299.9999	-1,300.0000	0.0001
Live	MY kN.m	-27,300.0012	-27,300.0000	-0.0012

Maximum absolute member discrepancies are taken over all corresponding ends or frame stations. P, V2 and M3 maxima may occur at different locations and do not form one concurrent design-action tuple.

Case	Member group	Max $ \Delta P $ kN	Max $ \Delta V2 $ kN	Max $ \Delta M3 $ kN.m
Dead	Piers	4.435	0.737	4.402
Dead	Spandrels	0.623	0.224	0.366
Dead	Frame beams	0.451	0.168	0.644
Dead	Frame columns	0.304	0.524	0.970
Live	Piers	69.551	19.359	89.334
Live	Spandrels	1.912	5.915	4.498
Live	Frame beams	8.796	5.091	17.460

Case	Member group	Max $ \Delta P $ kN	Max $ \Delta V_2 $ kN	Max $ \Delta M_3 $ kN.m
Live	Frame columns	37.912	11.860	21.002

For forces, absolute differences of 2 kN or less are treated as small differences; percentages around negligible reference values are not meaningful. The archived workbook retains its editable magnitude and difference floors. These are reporting rules, not solver-accuracy tolerances. Current full-precision results are in VerificationData-20261025.

F.6 Live Load Case Comparison

Live axial force comparison

The overturning response changes axial force from tension in the lower left pier to compression in the right pier. Both views show the same broad redistribution and the accumulation toward the base.

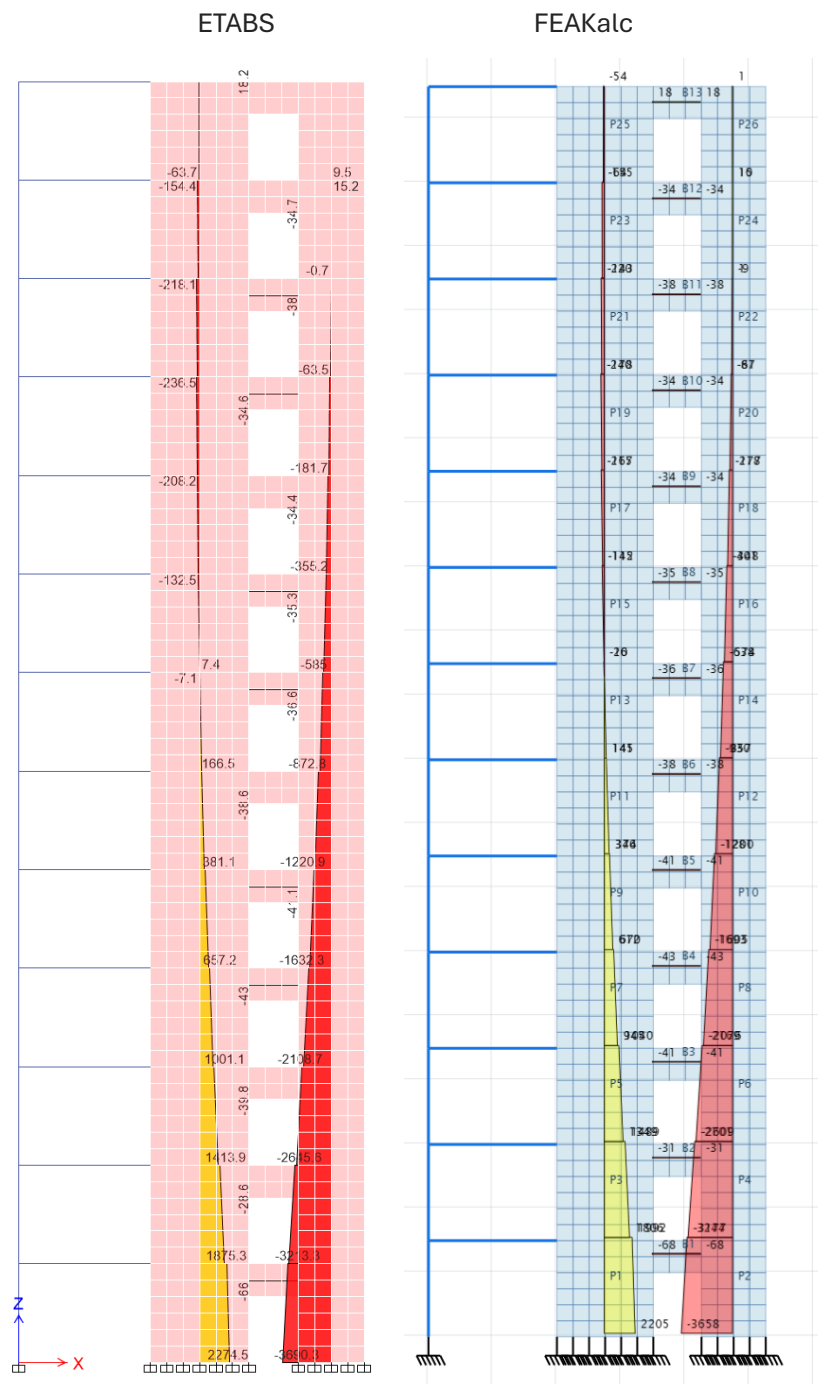


Figure 3. Axial force in kN. The supplied FEAkalc labels include both member ends; neighbouring labels overlap in places.

Live shear force comparison

Pier shear generally builds toward the base, while spandrel shear varies through the height.

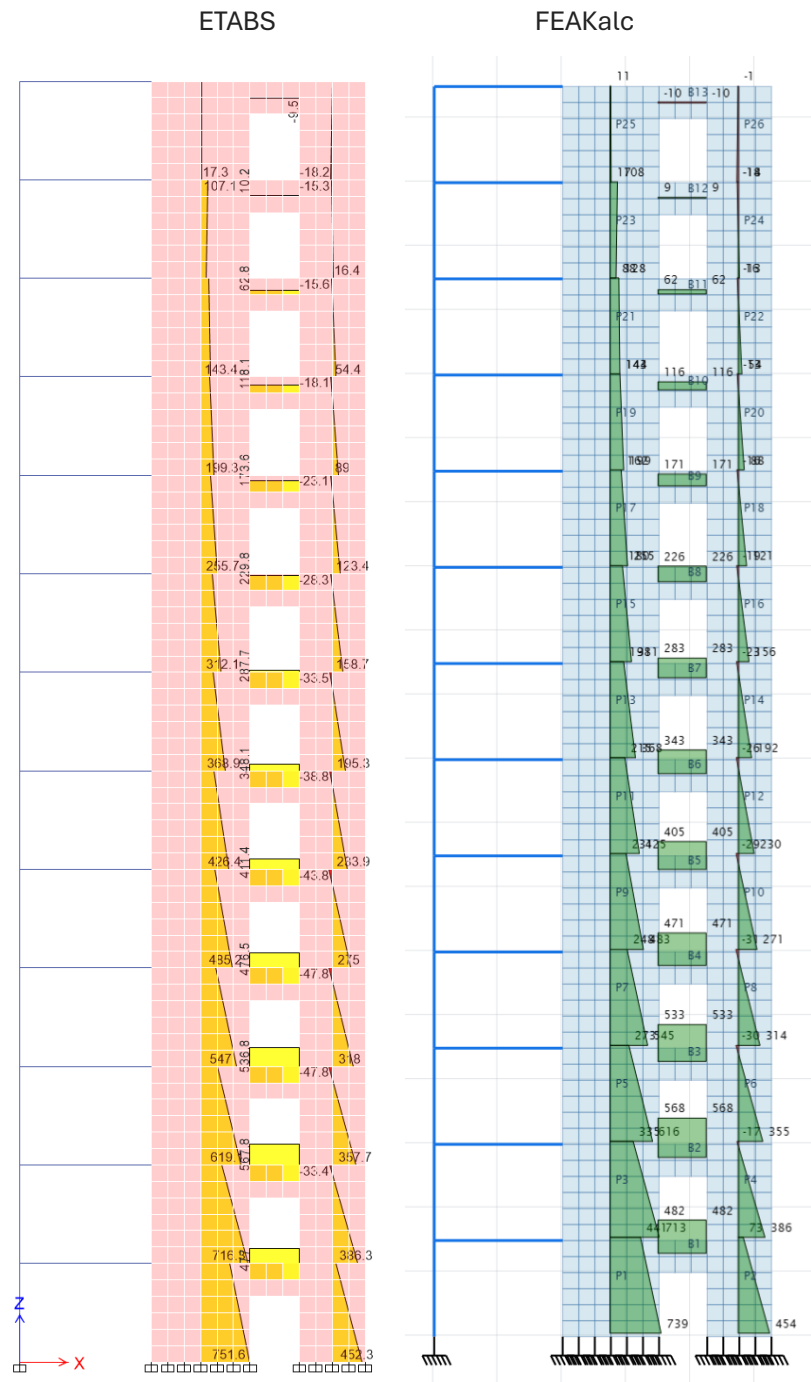


Figure 4. Local shear V2 in kN. ETABS and FEAkalc diagram fills use different colours; compare the labels and distribution.

Live bending moment comparison

Both programs show double-curvature spandrel moments and increasing pier demand toward the base. Sign changes in the upper pier regions occur in both graphical records.

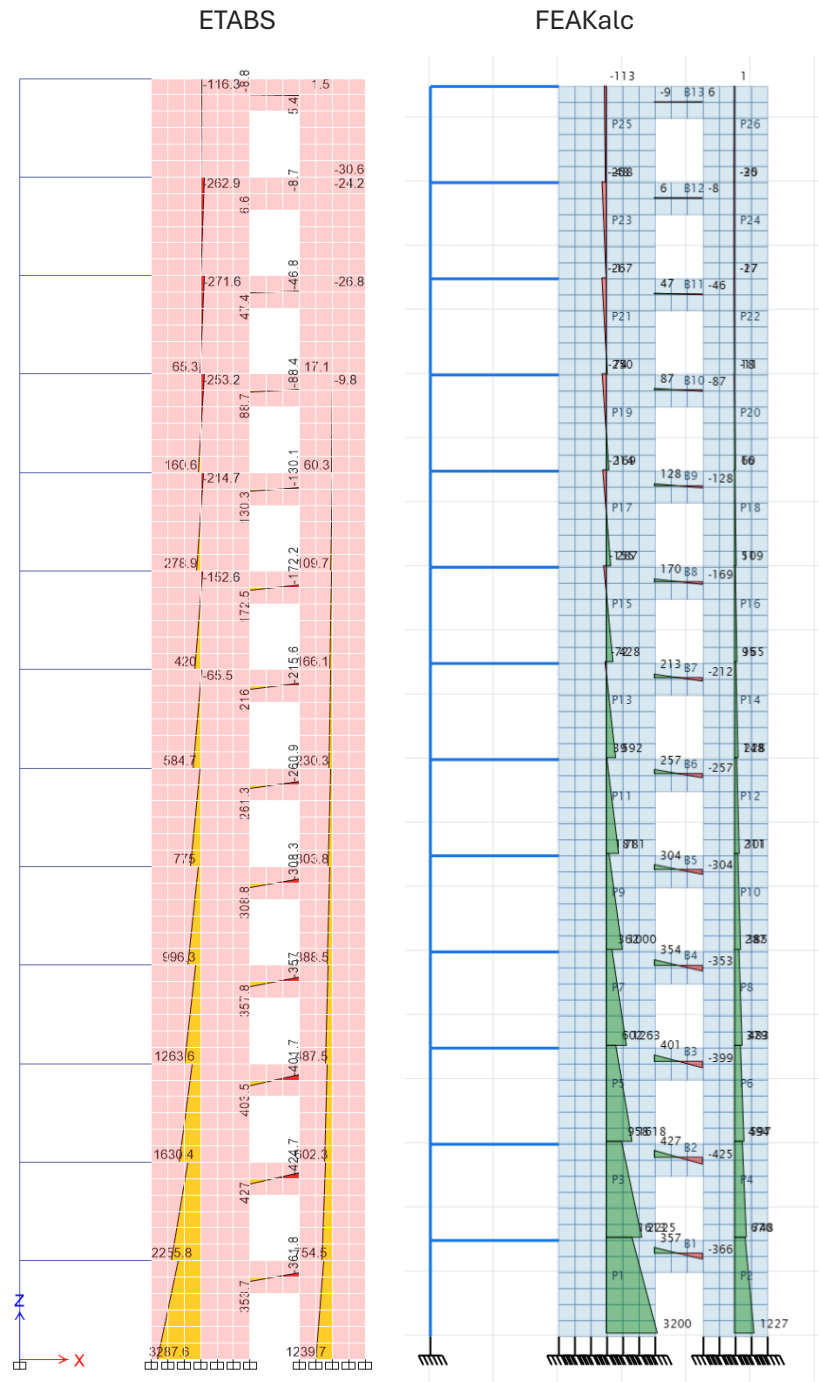


Figure 5. Local moment M_3 in kN.m. Relative plot widths are independently scaled in each application.

Live deformation comparison

Both deformed shapes sway in the direction of the applied horizontal forces and show a similar overall curvature. The FEAkalc screenshot uses a larger deformation magnification.

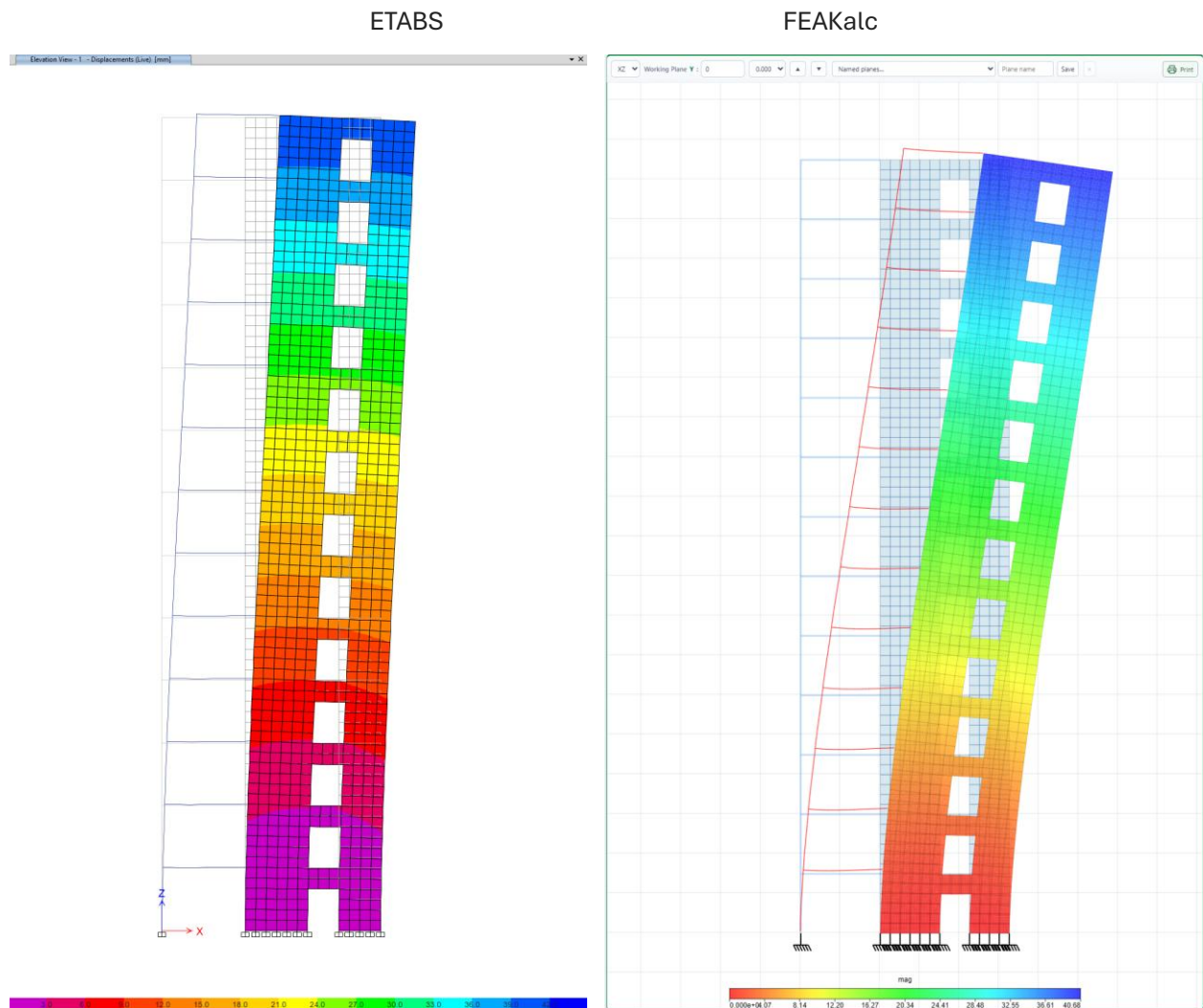


Figure 6. Supplied displacement displays from the earlier comparison. FEAkalc shows magnitude; colour ranges and deformation magnifications differ. Current roof translations are given in the numerical results table.

The screenshots are retained with their original legends and display conventions. Numerical comparison should use the same displacement component at the same coordinates, rather than the visual displacement amplitude or a colour match.

F.7 Dead Load Comparison (self-weight)

Dead axial force comparison

Self-weight produces compression increasing toward the base in both piers. The wider left pier attracts greater compression. Most upper spandrel axial values are close to zero.

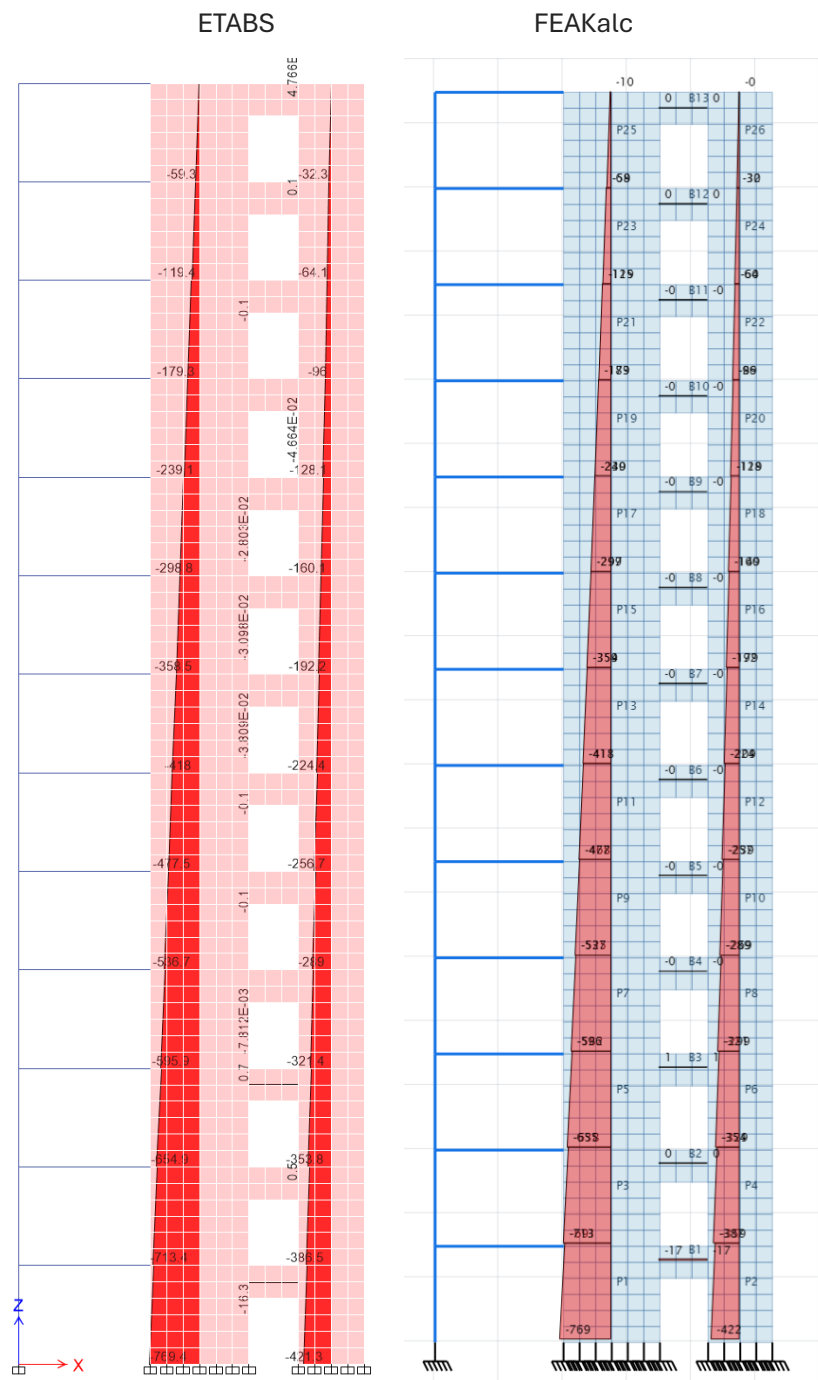


Figure 7. Axial force in kN under own weight. Rounded zero labels are retained and do not imply an exactly zero computed action.

Dead shear force comparison

Spandrel shear changes across the span under self-weight. Most pier shears are small, with larger local effects near the base. Absolute force differences are more informative than percentages around these small values.

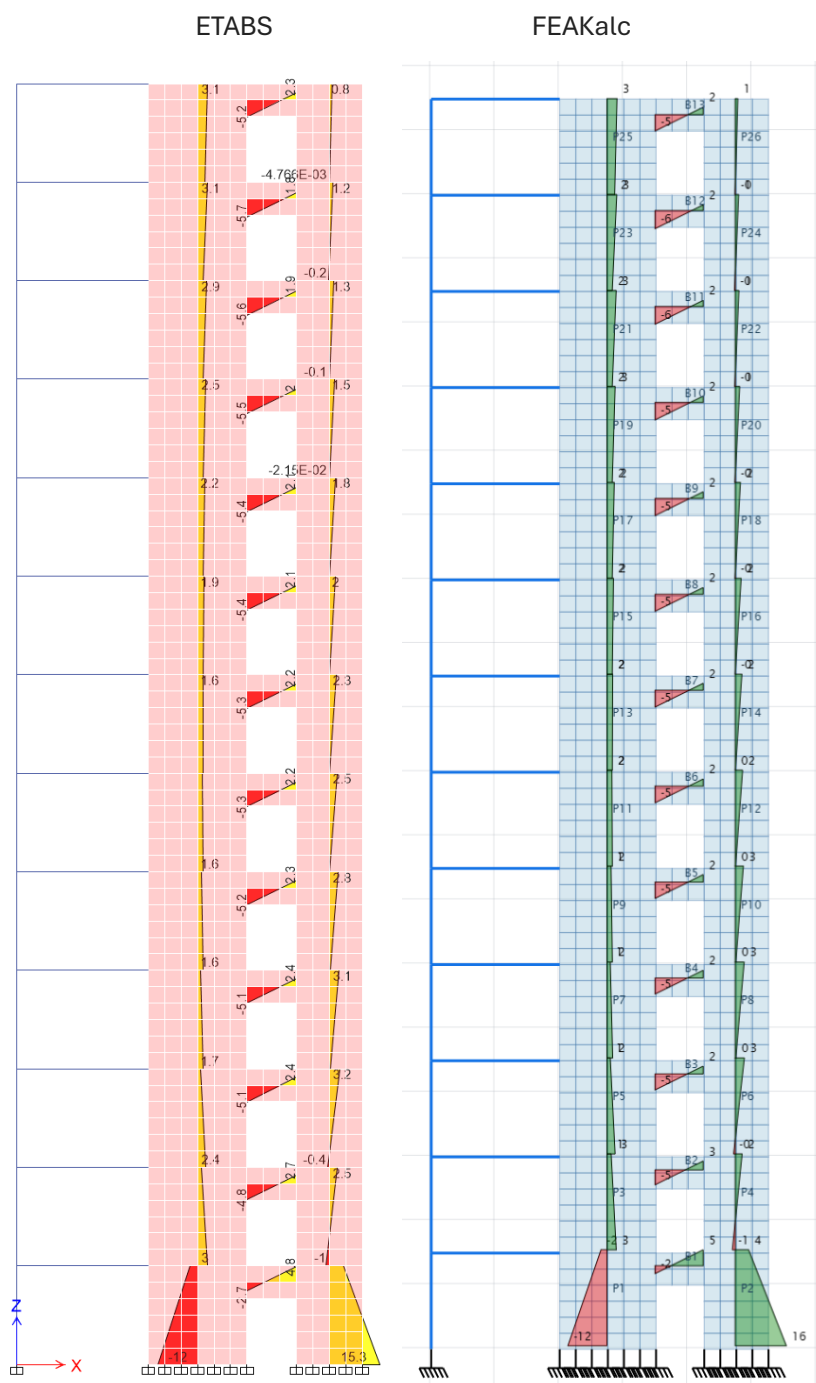


Figure 8. Local shear V2 in kN under own weight. Different end values are expected under distributed gravity loading.

Dead bending moment comparison

Both views show the same general pier-moment pattern under gravity, including the change toward the base and the smaller spandrel end actions. Differences remain visible in the lower pier values.

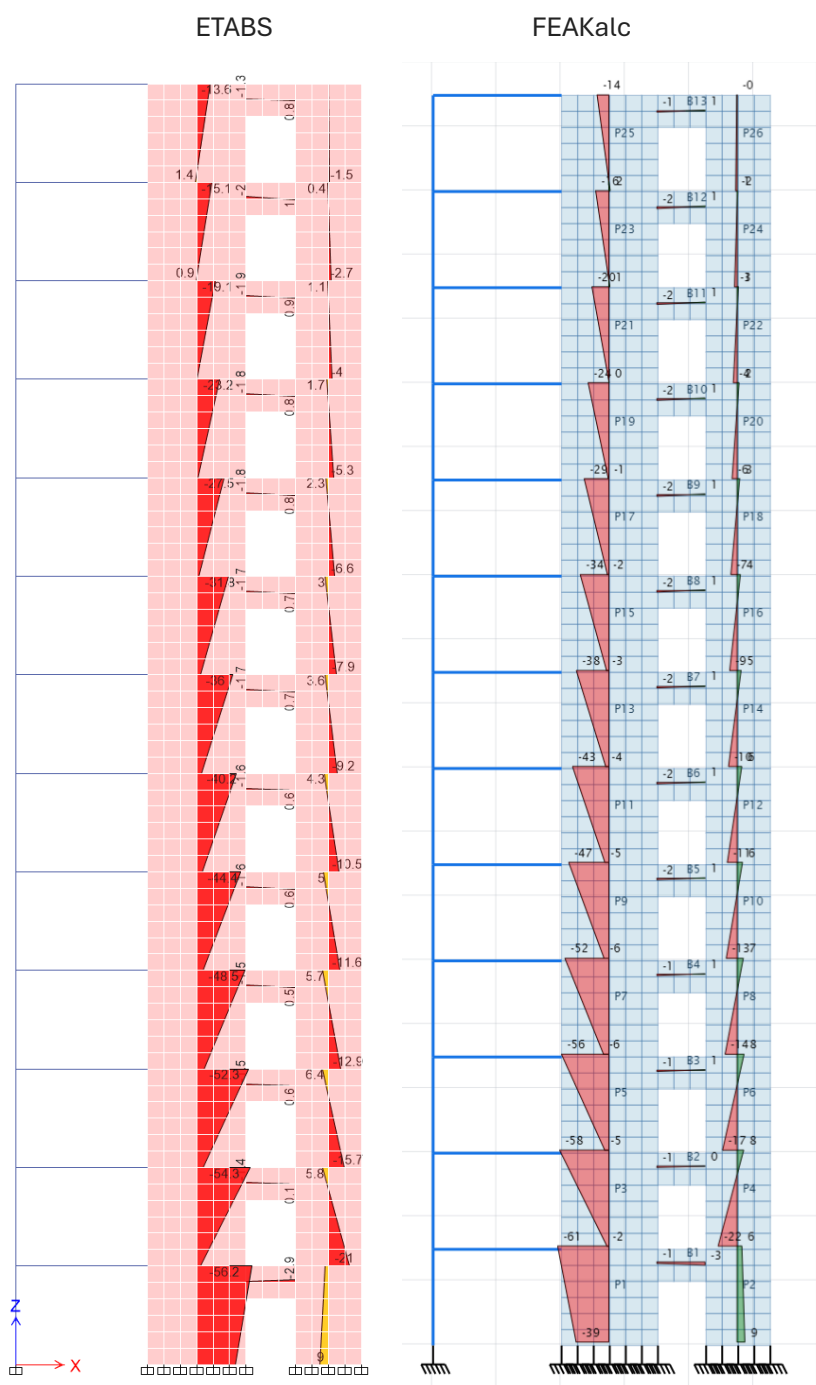


Figure 9. Local moment M_3 in kN.m under own weight. Labels are the supplied graphical output, not /8substituted numerical table values.

Dead deformation comparison

The deformed shapes show gravity shortening accompanied by leftward movement of the coupled assembly. Their on-screen exaggeration differs and must not be interpreted as a direct stiffness ratio.

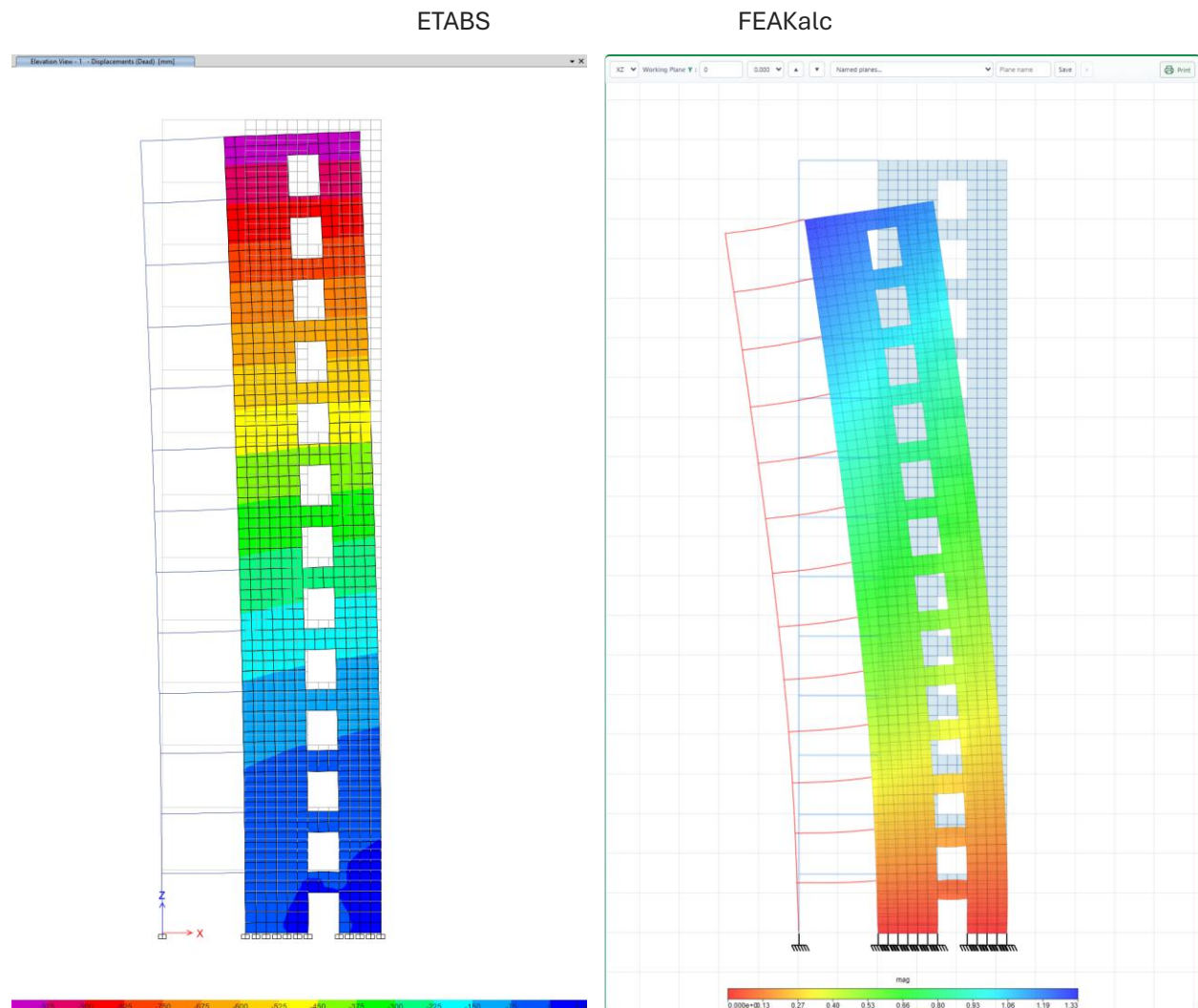


Figure 10. FEAkalc displays non-negative displacement magnitude. ETABS displays a signed component, as indicated by its negative colour legend. These contour colours do not represent the same quantity.

The screenshots are retained with their original legends and display conventions. Numerical comparison should use the same displacement component at the same coordinates, rather than the visual displacement amplitude or a colour match.

F.8 Applicability and remaining verification

The repaired automatic mesher reproduces the ETABS shell layout directly from the undivided wall model and eliminates the observed B10/B11 connectivity problem. Current Dead and horizontal Live analyses reproduce global equilibrium and the broad force and deformation patterns. This finding applies to the stated linear elastic benchmark.

Remaining differences include 3.22% in Live roof horizontal displacement and local frame actions. The matching shell topology now removes the earlier mesh-layout difference from this comparison. Shell interpolation, stiffness formulation, frame connections, local-axis conventions and force recovery still require separate investigation before assigning the residual differences to one cause.

Appendix G - Sample FEAKalc Design Memos

Pier P14 — section and in-plane shear design

AS 3600:2018 with Amendments 1 and 2: section and in-plane shear design. Enhanced seismic longitudinal reinforcement and fitment detailing remain separate. Response-spectrum component peaks are checked as the selected combination envelope actions, not as a time-concurrent response history. Boundary zones are screened from gross elastic top/bottom section stresses; intermediate heights and fitment detailing are not checked.

Combination coverage

Combination	Axial/flexural section	Shear
LC-1.35DL	Checked	Checked
LC-1.2DL+1.5LL	Checked	Checked
LC-AUTO-1 Dead + 1 RS1	Peak envelope checked	Checked
LC-AUTO-1 Dead + 0.6 Live + 1 RS1	Peak envelope checked	Checked
LC-AUTO-1 Dead - 1 RS1	Peak envelope checked	Checked
LC-AUTO-1 Dead + 0.6 Live - 1 RS1	Peak envelope checked	Checked
Section L × t / height	2,000 × 200 / 3,000 mm	
Concrete f'c / steel fy / fitment fys	40 / 500 / 500 MPa	
Reinforcement centre from face	54 mm	
Minimum longitudinal steel	0.25% = 1,000 mm ² total	
Provided longitudinal steel	14 N12; 1,583.36 mm ² ; 0.396%	
Spacing limits	Maximum min(2.5t, 350) = 350 mm; minimum clear 3db	
Design wall for out of plane buckling	Not selected — restraint assessed by the engineer; moment magnifier = 1.00	
Governing flexure / end	LC-1.2DL+1.5LL / Bottom	
Flexural utilisation	29.62%	
Overall utilisation	35.06%	

2,000 × 200 mm — 7 N12 each face



Centre from face: 54 mm; spacing along wall: 315.33 mm

Selected combination section actions — including spectrum peaks

Analysis P is positive in tension; section compression $N^* = -P$. M22 = integrated M2 (out of plane); M33 = integrated M3 (in plane). Spectrum rows are checked as the selected combination component-peak envelope, not as a time-concurrent response history. Static rows use their concurrent analysis actions.

Minimum compression eccentricities are 0.05t and 0.05L. In-plane magnification is 1.00.

Combination	End	P kN	M22 kNm	M33 kNm	Design M22	Design M33	δ_{out}	Util. %
LC-1.35DL	Bottom	-301.17	0	-17.88	3.01	30.12	1	8.11
LC-1.35DL	Top	-232.84	0	10.47	2.33	23.28	1	6.78
LC-1.2DL+1.5LL	Bottom	-1,484.59	0	308.4	14.85	308.4	1	29.62
LC-1.2DL+1.5LL	Top	-1,137.6	0	111.04	11.38	113.76	1	17.14
LC-AUTO-1 Dead + 1 RS1 Component-peak envelope	Bottom	-33.61	0	34.75	0.34	34.75	1	7.35
LC-AUTO-1 Dead + 1 RS1 Component-peak envelope	Top	-21.4	0	29.79	0.21	29.79	1	6.29
LC-AUTO-1 Dead + 0.6 Live + 1 RS1 Component-peak envelope	Bottom	-520.37	0	164.47	5.2	164.47	1	23.53
LC-AUTO-1 Dead + 0.6 Live + 1 RS1 Component-peak envelope	Top	-393.65	0	70.48	3.94	70.48	1	13.4
LC-AUTO-1 Dead - 1 RS1 Component-peak envelope	Bottom	-412.56	0	-61.24	4.13	61.24	1	12.3
LC-AUTO-1 Dead - 1 RS1 Component-peak envelope	Top	-323.54	0	-14.27	3.24	32.35	1	8.5
LC-AUTO-1 Dead + 0.6 Live - 1 RS1 Component-peak envelope	Bottom	-899.31	0	68.48	8.99	89.93	1	15.29
LC-AUTO-1 Dead + 0.6 Live - 1 RS1 Component-peak envelope	Top	-695.8	0	26.42	6.96	69.58	1	13.42

Governing section capacity

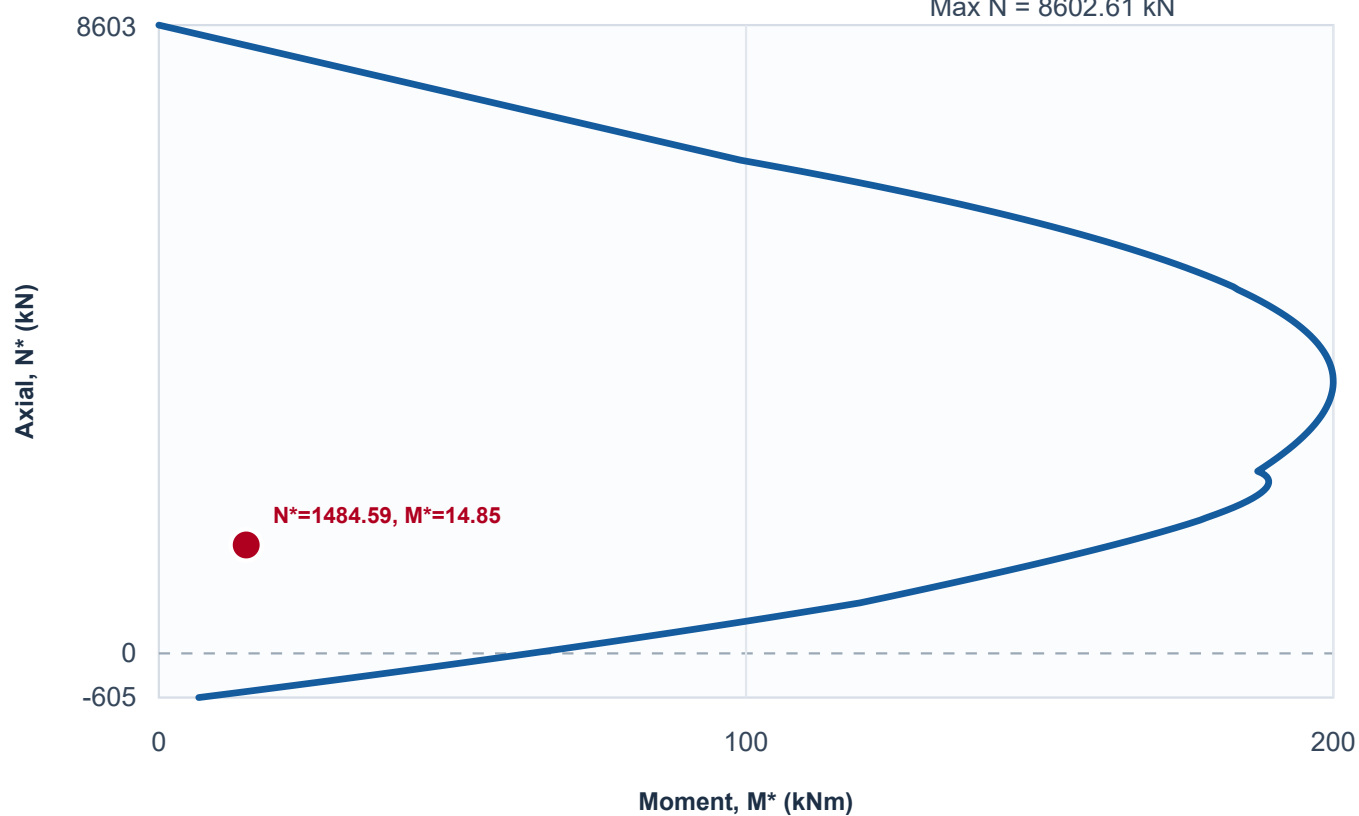
Moment capacities at $N^* = 1,484.59$ kN: $\phi\mu_{22} = 162.17$ kNm; $\phi\mu_{33} = 1,741.26$ kNm. Utilisation is the maximum of axial, uniaxial and applicable biaxial checks.

Biaxial check applies: both adopted design moments are nonzero. The biaxial check limits ϕ to 0.65; capacities at this N^* are 149.38 kNm (M22) and 1,566.73 kNm (M33), with exponent $\alpha = 1$.

Out of plane: N–M22

Max M = 199.55 kNm

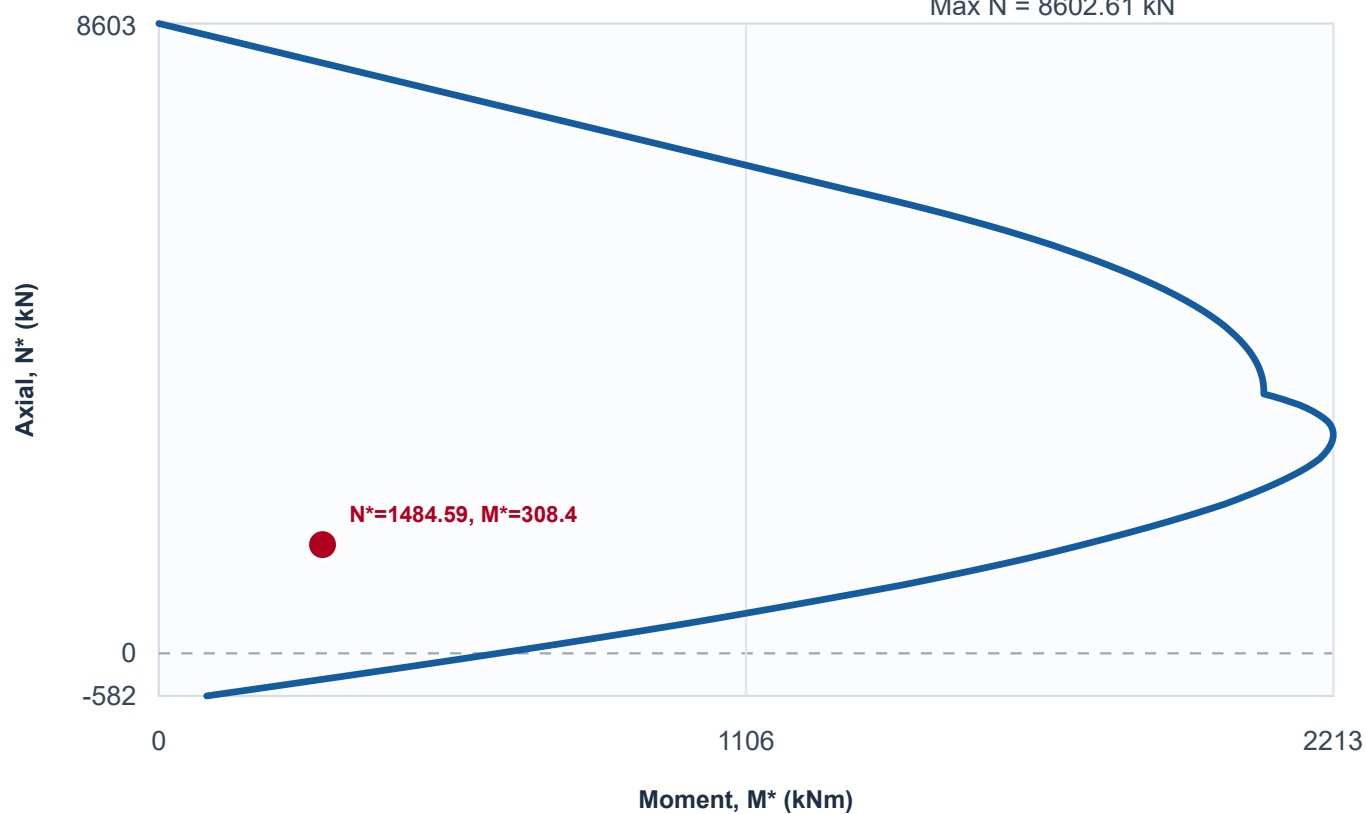
Max N = 8602.61 kN



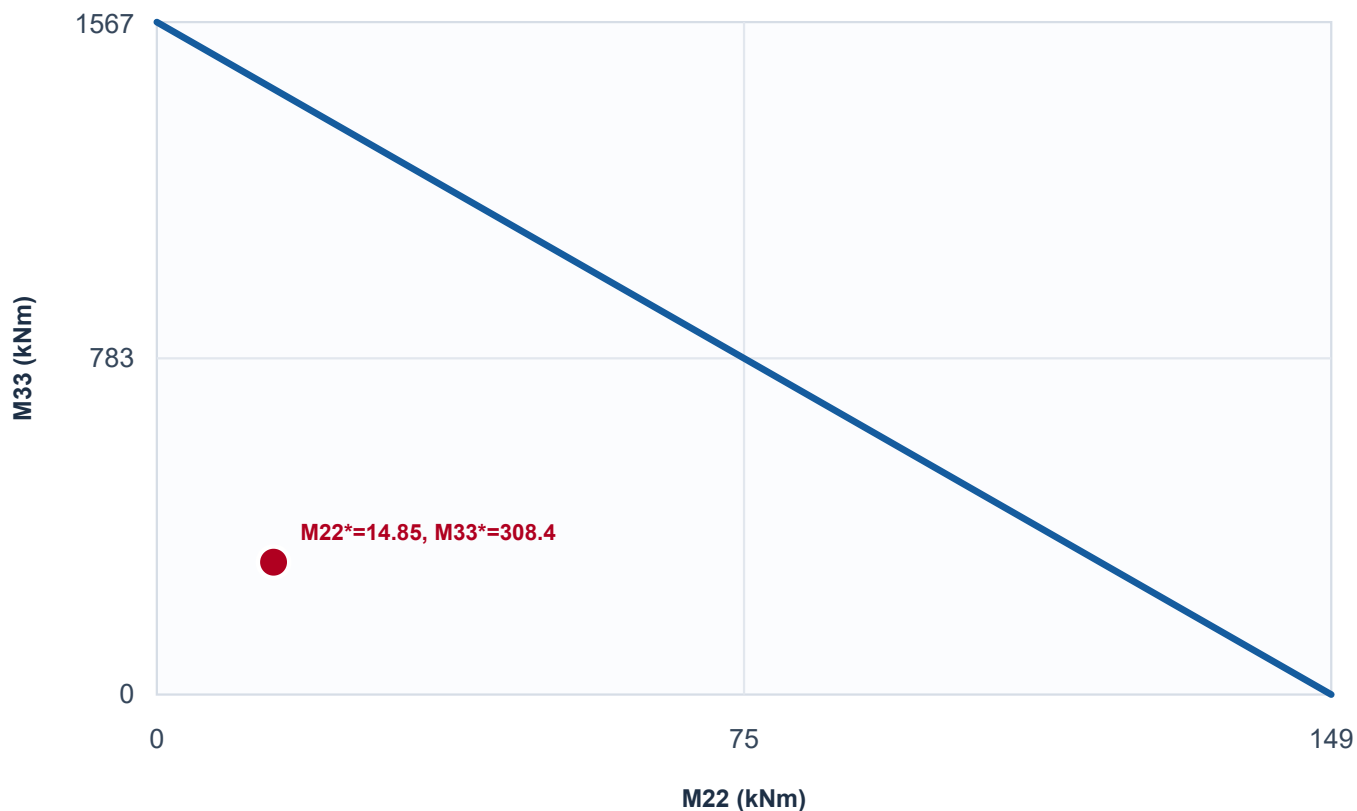
In plane: N–M33

Max M = 2212.64 kNm

Max N = 8602.61 kN



Biaxial at governing N*



In-plane shear and horizontal reinforcement — AS 3600:2018

Wall height H / length L / H/L	3,000 / 2,000 mm / 1.5
Concrete contribution V_{uc} (11.6.3)	505.96 kN
Provided horizontal reinforcement	N12 @ 350 mm EF; 646.27 mm ² /m total
Required / provided horizontal ratio	0.25% / 0.323%
Minimum / crack control (11.7)	0.25%; A1, Minor, restrained
Effective reinforcement ratio (11.6.4)	0.323%
Reinforcement contribution V_{us}	517.02 kN
Shear ϕ / web crushing ϕ	0.75 / 0.70
Web crushing $V_{u,max}$ / design limit	2,560 / 1,792 kN
Design capacity $\min[\phi(V_{uc}+V_{us}), 0.70V_{u,max}]$	767.24 kN
Governing shear	LC-1.2DL+1.5LL / Bottom; 35.06%; PASS

With $r = H/L$, $V_{uc} = (0.66 - 0.21r)\sqrt{f'_c}(0.8L_t)$ for $r \leq 1$. For $r > 1$, use the lesser of that expression and $[0.05 + 0.1/(r - 1)]\sqrt{f'_c}(0.8L_t)$, but not less than $0.17\sqrt{f'_c}(0.8L_t)$. $V_{us} = \rho_w f_{ys}(0.8L_t)$; $V_{u,max} = 0.2f'_c(0.8L_t)$. Required shear ratio = $\max[0, (V^*/\phi - V_{uc})/(f_{ys} 0.8L_t)]$. Calculations use N and mm; displayed forces are kN.

In-plane shear and horizontal minimum/crack-control reinforcement satisfied. The basic horizontal minimum is retained at 0.25%. For restrained walls, exposure and crack-control ratios may govern. $\phi = 0.75$ assumes Class N fitments meeting 8.2.1.7; the engineer must detail them accordingly. Web crushing always uses $\phi = 0.70$.

Combination	End	Original V2 kN	Spectrum VE kN	Factor Ω	Amplified V2 kN	Design V kN	Shear util. %
LC-1.35DL	Bottom	-0.39	0	1	-0.39	0.39	0.05
LC-1.35DL	Top	0.73	0	1	0.73	0.73	0.1
LC-1.2DL+1.5LL	Bottom	268.98	0	1	268.98	268.98	35.06
LC-1.2DL+1.5LL	Top	-3.62	0	1	-3.62	3.62	0.47
LC-AUTO-1 Dead + 1 RS1	Bottom	35.17	35.46	1.5	52.91	52.91	6.9
LC-AUTO-1 Dead + 1 RS1	Top	0.86	0.32	1.5	1.02	1.02	0.13
LC-AUTO-1 Dead + 0.6 Live + 1 RS1	Bottom	142.91	35.46	1.5	160.64	160.64	20.94
LC-AUTO-1 Dead + 0.6 Live + 1 RS1	Top	-0.85	0.32	1.5	-0.69	0.85	0.11
LC-AUTO-1 Dead - 1 RS1	Bottom	-35.75	-35.46	1.5	-53.49	53.49	6.97
LC-AUTO-1 Dead - 1 RS1	Top	0.22	-0.32	1.5	0.06	0.22	0.03
LC-AUTO-1 Dead + 0.6 Live - 1 RS1	Bottom	71.98	-35.46	1.5	54.24	71.98	9.38
LC-AUTO-1 Dead + 0.6 Live - 1 RS1	Top	-1.49	-0.32	1.5	-1.65	1.65	0.22

Spectrum shear envelope: $V_{\text{amplified}} = V_2 + (\Omega - 1) V_E$, where V_E is the signed, combination-factored response-spectrum contribution and $\Omega = 1.5$ is selected by the engineer. Static contributions retain their original combination factors. Both original and amplified shear magnitudes are checked. The μ , S_p and Earthquake selections used by Design piers do not add a second amplification here. One common reinforcement arrangement is selected against the section and shear envelopes of the selected combinations. Spectrum section inputs are labelled as component-peak envelopes, not as a time-concurrent response history. Top/bottom stations are checked without taking the near-base shear reduction in 11.6.1; intermediate sections remain subject to the analysis station coverage.

Method and scope

Existing AS 3600 concrete column section interaction engine, with a 0.25% wall minimum (11.7.1), two equal face rows and spacing checks (11.7.3). The smallest passing arrangement is selected from standard bars N12–N40 at or above the specified minimum diameter, with a 4% search ceiling. Tension retains its analysis sign. Curves are interpolated at the concurrent axial force.

Separate detailing checks: enhanced longitudinal reinforcement under 14.6.7, fitment detailing and confinement verification, out-of-plane shear, laps and anchorage remain separate. The out-of-plane buckling selection records the engineer's restraint assessment. The 2026 draft shear rules are not used.

Boundary-zone screening — REQUIRED

Building classification uses the engineer's rule: model vertical extent > 12 m means more than four storeys. Existing vertical reinforcement is assumed not already restrained to 10.7.4. Edge 1 = -local 2; Edge 2 = +local 2. Signed analysis N/M and gross rectangular properties are used, including the more compressed thickness face; section-design minimum eccentricities and moment magnifiers are not substituted. Only supplied top/bottom stations are screened. Zone lengths are enveloped for adoption over the labelled pier height; intermediate sections are not verified. Fitment sizes, spacing, anchorage and existing restraint are for the designer. All selected ULS combinations are screened; spectrum rows retain their component-peak-envelope qualification. The $0.3f_c$ axial-force provisions are not treated as local stress triggers. Critical tension reinforcement zones under 14.6.7 remain a separate check.

Model height 39 m: more than four storeys by the adopted 12 m rule. No fitment schedule is designed.

Edge 1 (-local 2): 0 mm

Edge 2 (+local 2): 11 mm



Adopted end-zone envelope over labelled pier; top/bottom checked only.
Shading indicates required zones, not a fitment arrangement.

Edge	Zone mm	Restraint category	Length governs	Category governs	Max compression MPa
1 (-local 2)	0	Not required by this screening			2.011
2 (+local 2)	11	Closed fitments to 14.6.2.3 / 10.7.4	LC-1.2DL+1.5LL Bottom	LC-1.2DL+1.5LL Bottom	6.024

Compression-positive stress: $\sigma(x) = -P/A + M33 \ x/I33 \pm M22(t/2)/I22$; the more compressed thickness face governs. $A = Lt$, $I33 = tL^3/12$, $I22 = Lt^3/12$. Lengths follow the $\sigma > 0.15f'_c$ crossing. For the low-rise deemed arrangement, adopted lengths are at least 1.5t (Figure 14.6.2.2). Above 0.20f'c, taller limited-ductile walls require 14.5.4 restraint; moderately ductile boundary elements require it regardless of stress. The strongest category is adopted throughout each enveloped edge zone.

Combination	Section	Edge 1 MPa	Edge 2 MPa	Edge 1 zone mm	Edge 2 zone mm	Category where required
LC-1.35DL	Bottom	0.887	0.619	0	0	No edge zone by this screening
LC-1.35DL	Top	0.504	0.661	0	0	No edge zone by this screening
LC-1.2DL+1.5LL	Bottom	1.398	6.024	0	11	Closed fitments to 14.6.2.3 / 10.7.4
LC-1.2DL+1.5LL	Top	2.011	3.677	0	0	No edge zone by this screening
LC-AUTO-1 Dead + 1 RS1 Component-peak envelope	Bottom	-0.177	0.345	0	0	No edge zone by this screening
LC-AUTO-1 Dead + 1 RS1 Component-peak envelope	Top	-0.17	0.277	0	0	No edge zone by this screening
LC-AUTO-1 Dead + 0.6 Live + 1 RS1 Component-peak envelope	Bottom	0.067	2.534	0	0	No edge zone by this screening
LC-AUTO-1 Dead + 0.6 Live + 1 RS1 Component-peak envelope	Top	0.456	1.513	0	0	No edge zone by this screening
LC-AUTO-1 Dead - 1 RS1 Component-peak envelope	Bottom	1.491	0.572	0	0	No edge zone by this screening
LC-AUTO-1 Dead - 1 RS1 Component-peak envelope	Top	0.916	0.702	0	0	No edge zone by this screening
LC-AUTO-1 Dead + 0.6 Live - 1 RS1 Component-peak envelope	Bottom	1.735	2.762	0	0	No edge zone by this screening
LC-AUTO-1 Dead + 0.6 Live - 1 RS1 Component-peak envelope	Top	1.541	1.938	0	0	No edge zone by this screening

Member B7	Governing combinations Shear: LC-1.2DL+1.5LL; flexure: LC-1.2DL+1.5LL	Node utilisation 87.9%
Shear steel 0.6268 mm²/mm	Flexural steel 831 mm²	Combinations checked 6

Nodal stresses within capacity; calculated reinforcement shown.

Geometry, materials and reinforcement

Property	Value	Property	Value
f'c (MPa)	40	Thickness (mm)	200
fy (MPa)	500	Edge node (mm)	160
Tie fys (MPa)	500	Mid node (mm)	250
Depth (mm)	1,000	Reo centre (mm)	80
Clear span (mm)	1,500	Overstrength factor	1.5

Design force model

Analysis forces are in kN and kN·m about the reported member axes. At each end, $V_{design} = V_{analysis} + (\Omega - 1) V_E$, where V_E is the signed, combination-factored response-spectrum contribution and $\Omega = 1.5$ is selected by the engineer. Static load-case contributions are unchanged. The shear envelope includes both the original and amplified combination demands.

Longitudinal tie reinforcement uses the original signed analysis M3 values, enveloped independently of shear. It is not divided by Ω . Working span $L_w = \text{clear span} + \text{edge node width}$. The shear-only strut-and-tie model adopts equal end moments in equilibrium, $M_L = M_R = V_{design} L_w / 2$, to calculate shear-tie forces and nodal stresses. These equivalent moments do not replace the analysis moments for longitudinal reinforcement.

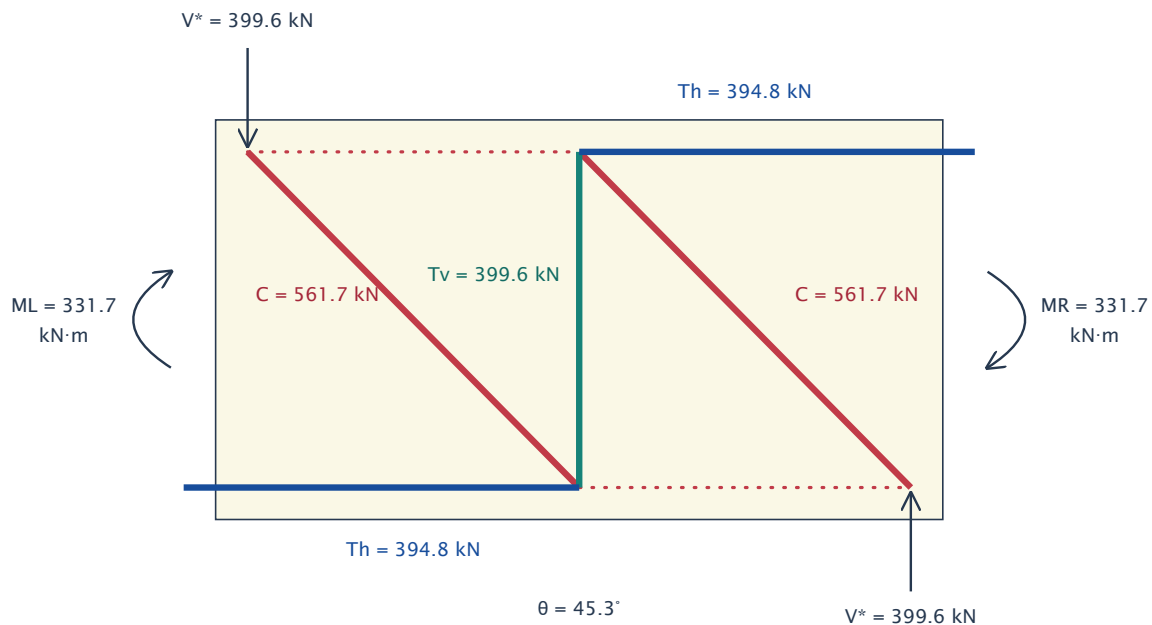
Response-spectrum actions are signed component envelopes. Shear and moment are checked separately; their component peaks are not treated as concurrent N-M-V actions. Axial-force interaction is outside this calculation.

Analysis forces and adopted design actions

Combination	Left V2 (kN)	Right V2 (kN)	Left M3 (kN·m)	Right M3 (kN·m)	Vdesign (kN)	Each shear-model M (kN·m)
LC-1.35DL	-7.5	2.6	-2.4	1.3	7.5	6.3
LC-1.2DL+1.5LL	390.6	399.6	295.9	-296.7	399.6	331.7
LC-AUTO-1 Dead + 1 RS1	47.4	55	38	40.7	81.5	67.7
LC-AUTO-1 Dead + 0.6 Live + 1 RS1	206.4	213.9	157.2	-78.4	240.4	199.6
LC-AUTO-1 Dead - 1 RS1	-58.6	-51.1	-41.6	-38.8	85.1	70.6
LC-AUTO-1 Dead + 0.6 Live - 1 RS1	100.3	107.8	77.6	-158	107.8	89.4

Combination	Left VE (kN)	Right VE (kN)	Left amplified V (kN)	Right amplified V (kN)	Left longitudinal As (mm²)	Right longitudinal As (mm²)
LC-1.35DL	0	0	-7.5	2.6	6.8	3.6
LC-1.2DL+1.5LL	0	0	390.6	399.6	828.9	831.2
LC-AUTO-1 Dead + 1 RS1	53	53.1	74	81.5	106.4	114
LC-AUTO-1 Dead + 0.6 Live + 1 RS1	53	53.1	232.9	240.4	440.4	219.7
LC-AUTO-1 Dead - 1 RS1	-53	-53.1	-85.1	-77.7	116.5	108.7
LC-AUTO-1 Dead + 0.6 Live - 1 RS1	-53	-53.1	73.8	81.2	217.4	442.5
Combination		Load case	Factor	Spectrum	Left V contribution (kN)	Right V contribution (kN)
LC-1.35DL		Dead	1.35	No	-7.5	2.6
LC-1.2DL+1.5LL		Dead	1.2	No	-6.7	2.3
LC-1.2DL+1.5LL		Live	1.5	No	397.3	397.3
LC-AUTO-1 Dead + 1 RS1		Dead	1	No	-5.6	1.9
LC-AUTO-1 Dead + 1 RS1		RS1	1	Yes	53	53.1
LC-AUTO-1 Dead + 0.6 Live + 1 RS1		Dead	1	No	-5.6	1.9
LC-AUTO-1 Dead + 0.6 Live + 1 RS1		Live	0.6	No	158.9	158.9
LC-AUTO-1 Dead + 0.6 Live + 1 RS1		RS1	1	Yes	53	53.1
LC-AUTO-1 Dead - 1 RS1		Dead	1	No	-5.6	1.9
LC-AUTO-1 Dead - 1 RS1		RS1	-1	Yes	-53	-53.1
LC-AUTO-1 Dead + 0.6 Live - 1 RS1		Dead	1	No	-5.6	1.9
LC-AUTO-1 Dead + 0.6 Live - 1 RS1		Live	0.6	No	158.9	158.9
LC-AUTO-1 Dead + 0.6 Live - 1 RS1		RS1	-1	Yes	-53	-53.1

Strut-and-tie calculation



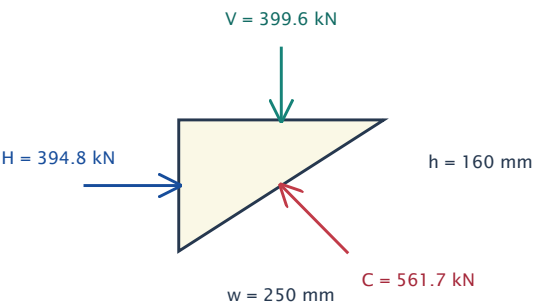
Green: central vertical shear tie T_v . Blue: horizontal flexural ties T_h . Red: two parallel diagonal compression struts C . Dotted red lines indicate the zero-force horizontal struts, $Ch = Th - C \cos \theta = 0$.

Calculation	Result
Working span $L_w = \text{span} + \text{edge node}$	1,660 mm
Truss depth $z = \text{depth} - 2 \times \text{reo centre}$	840 mm
Each equivalent shear-model end moment $M_L = M_R = V_{\text{design}} L_w / 2$	331.7 kN·m
Strut angle $\theta = \text{atan}(2z / L_w)$	45.3°
Vertical tie force (shear) $T_v = V_{\text{design}}$	399.6 kN
Total shear steel in the vertical tie $A_v = T_v \times 1000 / (0.85 f_{ys})$	940.2 mm ²
Horizontal tie force in the shear model $T_h = \max(M_L, M_R) / z$	394.8 kN
Diagonal strut force $C = T_v / \sin \theta$	561.7 kN
Horizontal strut $Ch = T_h - C \cos \theta$ (equal moments)	0 kN
Governing original analysis $ M_3 $	296.7 kN·m
Longitudinal tie force from analysis M_3 / z	353.2 kN
Longitudinal $A_s = M_3 \times 10^6 / (0.85 f_y z)$	831.2 mm ²
Required shear reinforcement $A_v/s = A_v / \text{clear span}$	0.6268 mm ² /mm

Nodal checks

Node	Width (mm)	Height (mm)	Stress (MPa)	Capacity (MPa)	Utilisation
Midspan CTT	250	160	12.338	14.04	87.9%
Edge CCT	160	160	12.487	18.72	66.7%
Edge CCC	160	160	12.487	23.4	53.4%

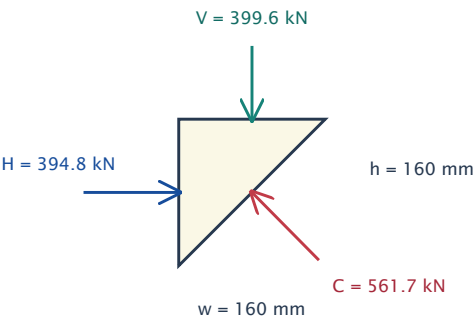
Midspan CTT



Midspan node; the opposite midspan node has the same demand in this equal-moment model.

Node geometry / check	Value
Width w	250 mm
Height h = 2 × reo centre	160 mm
Inclined face $\sqrt{(w^2 + h^2)}$	296.8 mm
Through-thickness	200 mm
Strut angle θ	45.3°
Applied nodal stress	12.338 MPa
Nodal capacity	14.04 MPa
Utilisation	87.9%

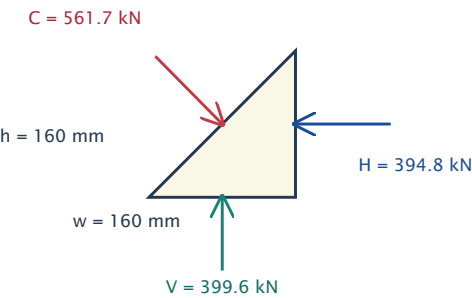
Edge CCT



Wall edge CCT node.

Node geometry / check	Value
Width w	160 mm
Height h = 2 × reo centre	160 mm
Inclined face $\sqrt{(w^2 + h^2)}$	226.3 mm
Through-thickness	200 mm
Strut angle θ	45.3°
Applied nodal stress	12.487 MPa
Nodal capacity	18.72 MPa
Utilisation	66.7%

Edge CCC



Opposite wall edge CCC node.

Node geometry / check	Value
Width w	160 mm
Height h = 2 × reo centre	160 mm
Inclined face $\sqrt{(w^2 + h^2)}$	226.3 mm
Through-thickness	200 mm
Strut angle θ	45.3°
Applied nodal stress	12.487 MPa
Nodal capacity	23.4 MPa
Utilisation	53.4%

Node drawings show the vertical resultant V, horizontal face resultant H and inclined strut force C. Dimensions are in mm and forces are in kN. For the adopted equal-moment model, $H = Th = C \cos \theta$.

Nodal capacity = $0.65 \times 0.9 \times f'_c \times \beta_n$, with $\beta_n = 0.6$ for the midspan CTT node, 0.8 for the edge CCT node and 1.0 for the edge CCC node. All three nodal checks contribute to the reported status. Shear reinforcement uses yield strength f_y ; bar diameter and spacing are selected during detailing.

Reinforcement quantities are required steel areas. Steel tension ties use $\phi = 0.85$ in accordance with Table 2.2.4 and Clause 7.3.2 of AS 3600:2018. The longitudinal tie area is based on the original analysis moment envelope, independently of the shear overstrength factor. Moment signs and the end areas govern reinforcement placement. PASS indicates that the reported nodal stresses do not exceed their capacities; it is not a declaration that all Section 14 requirements have been checked. Strut capacity and bursting reinforcement, anchorage, compatibility of the longitudinal reinforcement with the adopted shear-resisting mechanism, seismic detailing, out-of-plane actions and axial-force interaction require separate verification.

Analysis member

Member: B7. Wall panels: 1. Local axes (global XYZ): 1 = (1, 0, 0); 2 = (0, 0, 1); 3 = (0, -1, 0).

