FEAKalc 3D

Steel Design Manual to AS4100

Document Revision

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1. General Design Basis and Design Actions

1.1 General

The FEAKalc 3D steel design module performs strength and stability checks of structural steel frame members using the member actions obtained from the FEAKalc 3D finite element analysis.

The current steel design implementation is based on **AS 4100 – Steel Structures** and supports the following section families:

- I-sections;
- Parallel Flange Channels (PFC);
- Rectangular and Square Hollow Sections (RHS/SHS); and
- Circular Hollow Sections (CHS).

The design procedure operates directly on the analysed frame members. Each member is checked at the available analysis stations along its length, rather than being assessed only at the member ends.

For each station, the following design actions are transferred to the steel design module:

$$N^*,\quad M_{22}^*,\quad M_{33}^*,\quad V_2^*,\quad V_3^*,\quad T^*$$

where:

$$\begin{aligned}
N^* &= \text{design axial force}\\
M_{22}^* &= \text{design bending moment about the local 22 axis}\\
M_{33}^* &= \text{design bending moment about the local 33 axis}\\
V_2^* &= \text{design shear force in the local 2 direction}\\
V_3^* &= \text{design shear force in the local 3 direction}\\
T^* &= \text{design torsional action}
\end{aligned}$$

The member is checked for axial compression, bending about both principal axes, shear and combined axial force with biaxial bending. Torsion is currently transferred with the station actions but is not included as a separate utilisation check in the present design implementation.

1.2 Design Load Combinations

Steel design is performed for FEAKalc load combinations identified by the prefix:

$$\mathrm{LC}-$$

The program evaluates each applicable load combination independently.

For a general load combination consisting of n analysis load cases, an action F^* is obtained from:

$$\boxed{F^* = \sum_{i=1}^{n} \gamma_i F_i}$$

where:

- F_i = action obtained from analysis load case i;
- γ_i = load combination factor applied to load case i;
- F^* = resulting design action.

Accordingly, the axial force is:

$$\boxed{N^* = \sum_{i=1}^{n} \gamma_i \, N_i}$$

The bending moments are:

$$\boxed{M_{22}^* = \sum_{i=1}^{n} \gamma_i \, M_{22,i}}$$

$$\boxed{M_{33}^* = \sum_{i=1}^{n} \gamma_i \, M_{33,i}}$$

The shear forces are:

$$\boxed{V_2^* = \sum_{i=1}^{n} \gamma_i \, V_{2,i}}$$

$$\boxed{V_3^* = \sum_{i=1}^{n} \gamma_i \, V_{3,i}}$$

and the torsional action is:

$$\boxed{T^* = \sum_{i=1}^{n} \gamma_i \, T_i}$$

This combination is carried out independently at each analysis station. Therefore, the spatial distribution of the member actions produced by the finite element analysis is retained during design.

1.3 Design Stations

Let a frame member contain m design stations located at distances:

$$s_1, s_2, \dots, s_m$$

measured along the member.

The design action vector at station j may be written as:

$$\boxed{\boldsymbol{F}_j^* = [N_j^* \quad M_{22,j}^* \quad M_{33,j}^* \quad V_{2,j}^* \quad V_{3,j}^* \quad T_j^*]}$$

The steel design calculation is carried out independently for every station.

This is important for members subjected to varying bending moments, axial forces or shear forces because the critical design location is not assumed in advance.

The governing station is determined from the calculated utilisation rather than simply from the magnitude of one individual action.

1.4 Basic Capacity Relationship

The design procedure is expressed in terms of utilisation.

For an individual design action, the general form is:

$$\boxed{U = \frac{S^*}{\phi S_c}}$$

where:

- S^* = design action;
- S_c = calculated nominal member or section capacity;
- ϕ = capacity reduction factor;
- U = utilisation ratio.

The member satisfies the particular design check where:

$$\boxed{U \leq 1.0}$$

and fails that check where:

$$\boxed{U > 1.0}$$

The current implementation uses a capacity factor:

$$\boxed{\phi = 0.9}$$

for the principal steel member capacity calculations.

1.5 Individual Utilisation Checks

At each station, FEAKalc determines a number of utilisation ratios.

1.5.1 Axial compression

The axial utilisation takes the form:

$$\boxed{U_N = \frac{|N^*|}{\phi N_c}}$$

where N_c is the governing member compression capacity.

The calculation of N_c, including member slenderness and effective length effects, is discussed in the subsequent sections.

1.5.2 Major-axis bending

For bending about the major axis:

$$\boxed{U_{M33} = \frac{|M_{33}^*|}{\phi M_b}}$$

where M_b is the applicable major-axis member bending capacity.

For open sections, this capacity may be governed by lateral-torsional buckling.

1.5.3 Minor-axis bending

For bending about the minor axis:

$$\boxed{U_{M22} = \frac{|M_{22}^*|}{\phi M_{sy}}}$$

where M_{sy} represents the applicable minor-axis section bending capacity.

1.5.4 Shear

The general shear utilisation is:

$$\boxed{U_V = \frac{|V^*|}{\phi V_v}}$$

and is evaluated for the applicable local shear directions.

1.6 Combined Axial Force and Bending

In addition to the individual capacity checks, FEAKalc performs an interaction check combining axial force with bending about both principal axes.

The current implemented interaction relationship is of the form:

$$\boxed{U_I = U_N + U_{M33} + U_{M22}}$$

or:

$$\boxed{U_I = \frac{|N^*|}{\phi N_c} + \frac{|M_{33}^*|}{\phi M_b} + \frac{|M_{22}^*|}{\phi M_{sy}}}$$

The interaction utilisation therefore increases as the member simultaneously carries compression and bending about either principal axis.

The interaction requirement is:

$$\boxed{U_I \le 1.0}$$

The program reports this interaction separately from the individual axial, bending and shear checks.

1.7 Governing Station Utilisation

At each station, the governing utilisation is taken as the maximum of the calculated checks:

$$\boxed{U_j = \max[U_N, U_{M33}, U_{M22}, U_{V2}, U_{V3}, U_I]}$$

where j identifies the analysis station.

The governing utilisation for the complete member under a particular load combination is:

$$\boxed{U_{\text{member,LC}} = \max_{j=1,\dots,m} U_j}$$

The corresponding station is retained as the **governing station**.

1.8 Load Combination Envelope

FEAKalc repeats the station-by-station design calculation for each applicable `LC-` load combination.

If there are q design combinations, the final member utilisation is:

$$\boxed{U_{\text{member}} = \max_{k=1,\ldots,q} \left(U_{\text{member},k}\right)}$$

The load combination producing this maximum is reported as the:

$$\boxed{\text{Governing Load Combination}}$$

Thus, the final steel design result represents an envelope over both:

$$\boxed{\text{all member stations} \quad + \quad \text{all LC-* load combinations}}$$

The design output consequently identifies:

- member and section;
- PASS/FAIL status;
- maximum utilisation;
- governing load combination;
- governing station;
- governing design check;
- individual axial, bending and shear utilisations; and
- calculated member capacities.

A member is reported as satisfying the design checks when:

$$\boxed{U_{\text{member}} \leq 1.0}$$

and as failing where:

$$\boxed{U_{\text{member}} > 1.0}$$

This station-by-station and combination-by-combination envelope is explicitly reflected in the FEAKalc steel design workflow and result storage.

2 Effective Length, Member Restraint and Buckling Lengths

2.1 General

The compression capacity of a steel member depends not only on its cross-sectional resistance but also on its susceptibility to member buckling.

FEAKalc therefore determines effective member lengths independently about the local **22** and **33** axes:

$$\boxed{L_{e,22}}$$

and

$$\boxed{L_{e,33}}$$

These lengths are subsequently used in the steel member stability calculations.

The effective length calculation is based on the actual three-dimensional connectivity of the FEAKalc model. Collinear frame elements are first recognised as continuous member chains, after which the program identifies locations providing restraint to the relevant buckling direction.

The restraint calculation also considers intersections between frame members and shell elements.

2.2 Physical Member Length

For a frame element extending between points

$$P_1 = (x_1, y_1, z_1)$$

and

$$P_2 = (x_2, y_2, z_2)$$

the physical member length is:

$$\boxed{L = \sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2 + (z_2 - z_1)^2}}$$

The corresponding unit vector along the member axis is:

$$\boxed{\boldsymbol{e}_1 = \frac{1}{L}\begin{bmatrix} x_2 - x_1 \\ y_2 - y_1 \\ z_2 - z_1 \end{bmatrix}}$$

This defines the longitudinal direction of the frame element.

Together with the FEAKalc local 2 and local 3 directions, the member local coordinate system may be represented by:

$$\boxed{\{\boldsymbol{e}_1, \boldsymbol{e}_2, \boldsymbol{e}_3\}}$$

The local axes are important because restraint is assessed independently for the two member buckling directions.

2.3 Collinear Member Chains

A physical steel member may consist of several individual finite element frame objects.

FEAKalc therefore does not automatically assume that the length of an individual frame object is the buckling length.

Connected frame elements are first examined for collinearity.

For two connected members having unit direction vectors $\boldsymbol{e}_a$ and $\boldsymbol{e}_b$, the alignment condition implemented by FEAKalc is based on:

$$\boxed{|\boldsymbol{e}_a \cdot \boldsymbol{e}_b| \approx 1}$$

where:

$$\boldsymbol{e}_a \cdot \boldsymbol{e}_b = e_{ax}e_{bx} + e_{ay}e_{by} + e_{az}e_{bz}$$

The absolute value permits members defined in opposite node directions to be recognised as having the same physical alignment.

Connected and aligned frame elements are assembled into a continuous chain.

For a chain consisting of n elements:

$$L_1, L_2, \dots, L_n$$

the unrestrained chain length would be:

$$\boxed{L_{\text{chain}} = \sum_{i=1}^{n} L_i}$$

unless an intermediate restraint or shell intersection divides the chain into shorter effective segments.

2.4 Intermediate Frame Restraint

Connections to other non-collinear frame members are examined as potential intermediate restraints.

Importantly, FEAKalc does **not** automatically assume that every intersecting frame member provides restraint about both axes.

The orientation of the connected member relative to the local axes of the member being designed is considered.

Let:

$$\boldsymbol{d}_b$$

be the unit direction vector of the connected bracing member.

Its components in the local 2 and local 3 directions are:

$$\boxed{c_2 = |\boldsymbol{d}_b \cdot \boldsymbol{e}_2|}$$

and:

$$\boxed{c_3 = |\boldsymbol{d}_b \cdot \boldsymbol{e}_3|}$$

FEAKalc compares these components to determine which member direction is predominantly restrained.

For the 22 restraint calculation, the implemented criterion is:

$$\boxed{c_2 > c_3 \quad \text{and} \quad c_2 > 0.35}$$

For the 33 restraint calculation:

$$\boxed{c_3 > c_2 \quad \text{and} \quad c_3 > 0.35}$$

A connected member satisfying the applicable criterion creates a restraint node for that buckling direction.

This allows FEAKalc to determine different effective lengths for the two principal directions:

$$\boxed{L_{e,22} \neq L_{e,33}}$$

where the three-dimensional framing arrangement provides different restraint conditions about the two axes.

2.5 Axis-Specific Unrestrained Segments

Once the restraint nodes have been identified, each collinear chain is divided independently for the two buckling directions.

For the local 22 direction:

$$\boxed{L_{22,i} = s_{22,i+1} - s_{22,i}}$$

where $s_{22,i}$ are consecutive restraint positions along the chain.

Similarly, for the local 33 direction:

$$\boxed{L_{33,i} = s_{33,i+1} - s_{33,i}}$$

The effective length assigned to a member corresponds to the applicable unrestrained segment containing that member.

Consequently, the same physical frame element may have:

$$L_{e,22} = L_a$$

and:

$$L_{e,33} = L_b$$

with:

$$L_a \neq L_b$$

depending on the surrounding framing system.

2.6 Shell Elements as Restraint

FEAKalc also considers shell elements when determining member restraint.

This is particularly important for structural arrangements such as:

- columns intersecting floor slabs;
- columns passing through floor plates;
- steel members connected to wall or shell systems; and
- frame elements for which a shell intersection occurs between the member end nodes.

The program determines whether the member line intersects the plane of a shell.

The frame line may be written parametrically as:

$$\boxed{\boldsymbol{p}(t) = \boldsymbol{p}_1 + t(\boldsymbol{p}_2 - \boldsymbol{p}_1)}$$

where:

$$0 \le t \le 1$$

Let the shell plane be defined by a point $\boldsymbol{p}_s$ and unit normal vector $\boldsymbol{n}_s$.

The intersection parameter is obtained from:

$$\boxed{t = \frac{(\boldsymbol{p}_s - \boldsymbol{p}_1) \cdot \boldsymbol{n}_s}{(\boldsymbol{p}_2 - \boldsymbol{p}_1) \cdot \boldsymbol{n}_s}}$$

provided:

$$\boxed{(\boldsymbol{p}_2 - \boldsymbol{p}_1) \cdot \boldsymbol{n}_s \ne 0}$$

If:

$$0 \le t \le 1$$

the resulting intersection point is:

$$\boxed{\boldsymbol{p}_I = \boldsymbol{p}_1 + t(\boldsymbol{p}_2 - \boldsymbol{p}_1)}$$

FEAKalc then checks whether $\boldsymbol{p}_I$ lies within the actual shell polygon rather than merely within the infinite shell plane.

2.7 Shell Intersections Within a Member

Where a valid shell intersection occurs within the member rather than at its end, the intersection is treated as a break in the effective unrestrained length.

The distance of the intersection from the member start is:

$$\boxed{s_I = L\,t}$$

For example, consider a continuous member of length L intersected by a floor shell at:

$$s = s_I$$

The resulting segments are:

$$L_1 = s_I$$

and:

$$L_2 = L - s_I$$

The relevant unrestrained length is then established from the resulting segments rather than simply using the original full member length.

For several restraint positions:

$$0 = s_0 < s_1 < s_2 < \cdots < s_n = L$$

the individual unrestrained lengths are:

$$\boxed{L_i = s_i - s_{i-1}}$$

This treatment permits a shell passing through the interior of a frame object to provide an intermediate restraint even where the finite element itself has not explicitly been divided at the shell level.

2.8 Shell Restraint at Member Nodes

Shell intersections occurring at frame member nodes are also recognised by the implemented effective-length routine.

If the shell intersection occurs sufficiently close to the member start:

$$t \approx 0$$

the start node is identified as shell-braced.

Similarly, where:

$$t \approx 1$$

the end node is identified as shell-braced.

These shell-braced nodes are added to the restraint nodes used when splitting continuous collinear frame chains.

Thus, for a multi-storey column represented by several connected frame elements, floor shells may effectively divide the continuous column chain into storey-height buckling segments.

2.9 Automatically Determined Effective Length

Following the frame restraint and shell intersection calculations, FEAKalc obtains automatically determined effective lengths:

$$\boxed{L_{22,\text{auto}}}$$

and:

$$\boxed{L_{33,\text{auto}}}$$

These values represent the effective unrestrained lengths inferred from the model geometry and connectivity.

In the absence of a recognised intermediate restraint, the corresponding physical member or continuous chain length governs.

The automatically determined values are retained in the design data separately from any user-defined override.

2.10 Effective Length Factors

FEAKalc permits the automatically determined effective lengths to be replaced by user-defined effective length factors.

For the local 22 direction, where a factor K_{22} is specified:

$$\boxed{L_{e,22} = K_{22}L}$$

Similarly:

$$\boxed{L_{e,33} = K_{33}L}$$

where:

- L = physical frame member length;

- K_{22} = user-defined effective length factor for the 22 direction;
- K_{33} = user-defined effective length factor for the 33 direction.

The factors reported by the program may therefore be expressed as:

$$\boxed{\beta_{22} = \frac{L_{e,22}}{L}}$$

and:

$$\boxed{\beta_{33} = \frac{L_{e,33}}{L}}$$

For a user-defined factor:

$$\beta_{22} = K_{22}$$

and:

$$\beta_{33} = K_{33}$$

2.11 Direct Effective Length Override

The user may alternatively specify the effective length directly.

For the local 22 axis:

$$\boxed{L_{e,22} = L_{22,\text{user}}}$$

and for the local 33 axis:

$$\boxed{L_{e,33} = L_{33,\text{user}}}$$

The effective-length selection implemented by FEAKalc can therefore be represented as:

$$\boxed{L_{e,i} = \begin{cases} L_{i,\text{user}}, & \text{direct length override} \\ K_i L, & \text{effective length factor} \\ L_{i,\text{auto}}, & \text{automatic determination} \end{cases}}$$

for:

$$i = 22,\ 33$$

The user-defined value therefore takes precedence over the automatically calculated value.

Clearing the override returns the member to the automatic effective-length calculation.

2.12 Lateral-Torsional Buckling Restraint Length

A separate restraint length is provided for lateral-torsional buckling of members subjected to major-axis bending.

This length is denoted:

$$\boxed{L_{LTB}}$$

and is independent of:

$$L_{e,22} \quad \text{and} \quad L_{e,33}$$

because the physical restraint controlling lateral-torsional buckling need not be the same as that controlling flexural buckling under axial compression.

Where a user-defined LTB length is provided:

$$\boxed{L_{LTB} = L_{LTB,\mathrm{user}}}$$

Where no LTB override is specified, the value is left for determination by the steel design procedure.

This distinction is important:

$$\boxed{\text{Compression buckling: } L_{e,22},\ L_{e,33}}$$

whereas:

$$\boxed{\text{Lateral-torsional buckling: } L_{LTB}}$$

2.13 Effective Length Summary

The effective-length process used by FEAKalc may therefore be summarised as:

$$\boxed{\text{Frame Geometry} \rightarrow \text{Collinear Chains} \rightarrow \text{Frame/Shell Restraints} \rightarrow \text{Axis-Specific Segments} \rightarrow L_{22,\mathrm{auto}}, L_{33,\mathrm{auto}}}$$

followed by application of any user override:

$$\boxed{L_{22,\mathrm{auto}} \rightarrow \{L_{22,\mathrm{auto}}, K_{22}L, L_{22,\mathrm{user}}\} \rightarrow L_{e,22}}\ \boxed{L_{33,\mathrm{auto}} \rightarrow \{L_{33,\mathrm{auto}}, K_{33}L, L_{33,\mathrm{user}}\} \rightarrow L_{e,33}}$$

The resulting effective lengths are then passed to the steel design engine for calculation of member slenderness and compression capacity.

3 Section Slenderness, Classification and Effective Section Properties

3.1 General

The resistance of a steel section is influenced by the slenderness of its constituent plate elements. Local buckling may prevent a slender web or flange from developing the full cross-sectional resistance.

FEAKalc therefore evaluates the slenderness of the section before calculating compression and bending capacities.

The design engine distinguishes between:

- I-sections;
- PFC sections;
- RHS and SHS sections; and
- CHS sections.

For I-sections and PFCs, separate web and flange slenderness parameters are calculated. The resulting slenderness is used both in determining the effective area factor k_f for compression and in determining the effective section modulus for bending.

Designer

3.2 Yield-Stress-Modified Slenderness

The plate slenderness calculations used by FEAKalc include a yield-stress modification relative to a reference stress of:

$$250 \text{ MPa}$$

The general form is:

$$\boxed{\lambda = \frac{b}{t}\sqrt{\frac{f_y}{250}}}$$

where:

- b = relevant plate width;
- t = plate thickness;
- f_y = yield stress applicable to the plate element;
- λ = modified plate slenderness.

Separate yield stresses may be used for the web and flange:

$$f_{yw}$$

and:

$$f_{yf}$$

This is particularly relevant to I-sections, where the steel grade adopted by the design engine may depend on flange and web thickness.

3.3 I-Section Web Slenderness

For an I-section, FEAKalc takes the clear web depth as:

$$\boxed{d_w = d - 2t_f}$$

where:

- d = overall section depth;
- t_f = flange thickness.

The web slenderness is then calculated as:

$$\boxed{\lambda_w = \frac{d - 2t_f}{t_w}\sqrt{\frac{f_{yw}}{250}}}$$

where t_w is the web thickness.

The calculated value is retained in the design results as:

$$\lambda_{\text{web}}$$

and is reported in the FEAKalc steel design memo.

Designer

3.4 I-Section Flange Slenderness

For an I-section, the outstanding flange width on each side of the web is taken as:

$$\boxed{b_o = \frac{b_f - t_w}{2}}$$

The flange slenderness is therefore:

$$\boxed{\lambda_f = \frac{(b_f - t_w)/2}{t_f}\sqrt{\frac{f_{yf}}{250}}}$$

or:

$$\boxed{\lambda_f = \frac{b_f - t_w}{2t_f}\sqrt{\frac{f_{yf}}{250}}}$$

where:

- b_f = flange width;
- t_f = flange thickness;
- t_w = web thickness;

- f_{yf} = flange yield stress.

The web and flange slenderness parameters are subsequently used independently to determine which plate element controls the effective section properties.

Designer

3.5 Effective Area for Compression

Local plate slenderness also affects the compression resistance of the section.

FEAKalc determines an effective cross-sectional area:

$$A_e$$

and defines the section form factor:

$$\boxed{k_f = \frac{A_e}{A_g}}$$

where:

- A_g = gross cross-sectional area;
- A_e = effective area allowing for slender plate elements.

The value is limited to:

$$\boxed{0 \leq k_f \leq 1}$$

A fully effective section therefore has:

$$\boxed{k_f = 1.0}$$

while:

$$k_f < 1.0$$

indicates a reduction in the effective area due to plate slenderness.

The calculated k_f is subsequently used in the member compression calculation.

Designer

3.6 Effective Web and Flange Dimensions — I-Sections

For compression, FEAKalc determines effective web and flange dimensions from the calculated slenderness.

For a rolled I-section, the limiting values used by the implementation are:

$$\boxed{\lambda_{w,\lim} = 45}$$

and:

$$\boxed{\lambda_{f,\lim} = 16}$$

For a welded I-section, the corresponding values are:

$$\boxed{\lambda_{w,\lim} = 35}$$

and:

$$\boxed{\lambda_{f,\lim} = 14}$$

The effective web depth is calculated from:

$$\boxed{d_e = (d - 2t_f)\frac{\lambda_{w,\lim}}{\lambda_w}}$$

subject to:

$$\boxed{d_e \leq d - 2t_f}$$

Similarly, the effective flange dimension is calculated as:

$$\boxed{b_e = (b_f - t_w)\frac{\lambda_{f,\lim}}{\lambda_f}}$$

subject to:

$$\boxed{b_e \leq b_f - t_w}$$

The effective area used by FEAKalc is then:

$$\boxed{A_e = A_g - \left[(d - 2t_f) - d_e\right]t_w - 2\left[(b_f - t_w) - b_e\right]t_f}$$

and hence:

$$\boxed{k_f = \frac{A_e}{A_g}}$$

This formulation is implemented directly in the I-section design routine.

Designer

3.7 Section Classification for Bending

For bending resistance, FEAKalc classifies the section as:

$$\boxed{\text{Compact}}$$
$$\boxed{\text{Non-Compact}}$$

or:

$$\boxed{\text{Slender}}$$

The classification determines the effective section modulus Z_e used to calculate section moment capacity.

The general relationship is:

$$\boxed{M_s = \phi Z_e f_y}$$

with:

$$\boxed{\phi = 0.9}$$

in the current implementation.

Thus:

$$\boxed{M_s = 0.9 Z_e f_y}$$

The important point is that FEAKalc does **not** simply use the tabulated elastic or plastic modulus for every section. The modulus is modified according to the governing plate slenderness.

3.8 Compact Section Effective Modulus

For a compact section, FEAKalc permits the effective modulus to increase from the elastic section modulus towards the plastic modulus.

For major-axis bending:

$$\boxed{Z_{e,x} = \min(S_x, 1.5Z_x)}$$

where:

- Z_x = elastic section modulus;
- S_x = plastic section modulus.

Similarly, for minor-axis bending:

$$\boxed{Z_{e,y} = \min\left(S_y, 1.5Z_y\right)}$$

Thus the effective modulus cannot exceed either the tabulated plastic modulus or:

$$1.5Z$$

3.9 Non-Compact Section Effective Modulus

For a non-compact plate element, FEAKalc interpolates between the elastic modulus and the compact-section effective modulus.

Let:

$$Z_p = \min(S, 1.5Z)$$

Then the implemented interpolation has the general form:

$$\boxed{Z_e = Z + \frac{\lambda_n - \lambda}{\lambda_n - \lambda_p}\left(Z_p - Z\right)}$$

where:

- λ = calculated plate slenderness;
- λ_p = compact slenderness limit;
- λ_n = non-compact slenderness limit;
- Z = elastic section modulus;
- Z_p = effective modulus available to a compact section.

At:

$$\lambda = \lambda_p$$

the expression gives:

$$Z_e = Z_p$$

while at:

$$\lambda = \lambda_n$$

it gives:

$$Z_e = Z$$

The transition from compact to non-compact behaviour is therefore continuous.

3.10 Slender Section Effective Modulus

Where the governing plate element is slender, FEAKalc reduces the elastic modulus in proportion to the limiting slenderness.

The implemented relationship takes the form:

$$\boxed{Z_e = Z\frac{\lambda_n}{\lambda}}$$

where:

$$\lambda > \lambda_n$$

Consequently:

$$Z_e < Z$$

for a slender section.

This reduction represents the loss of effective bending resistance resulting from local buckling of the governing plate element.

3.11 Major-Axis Classification of I-Sections

For major-axis bending, FEAKalc first determines whether the web or flange controls the section classification.

The comparison used in the implementation is:

$$\boxed{\frac{\lambda_w}{115} > \frac{\lambda_f}{16}}$$

If this condition is satisfied, the web governs.

Otherwise, the flange governs.

Designer

3.11.1 Web-controlled major-axis classification

For the web:

$$\boxed{\lambda_w < 82 \quad \Rightarrow \quad \text{Compact}}$$

For the intermediate range:

$$\boxed{82 \leq \lambda_w < 115 \quad \Rightarrow \quad \text{Non-Compact}}$$

and:

$$\boxed{\lambda_w \geq 115 \quad \Rightarrow \quad \text{Slender}}$$

For a compact web:

$$\boxed{Z_{e,x} = \min(S_x, 1.5Z_x)}$$

For a non-compact web:

$$\boxed{Z_{e,x} = Z_x + \frac{115 - \lambda_w}{115 - 82}[\min(S_x, 1.5Z_x) - Z_x]}$$

For a slender web:

$$\boxed{Z_{e,x} = Z_x \frac{115}{\lambda_w}}$$

3.12 Flange-Controlled Major-Axis Classification

Where the flange controls, the compact and non-compact reference slenderness values used by the I-section routine are:

$$\boxed{\lambda_{fp} = 9}$$

and:

$$\boxed{\lambda_{fn} = 16}$$

For the compact range:

$$\lambda_f < 9$$

FEAKalc adopts:

$$\boxed{Z_{e,x} = \min(S_x, 1.5Z_x)}$$

For the non-compact range, the effective modulus is obtained by interpolation:

$$\boxed{Z_{e,x} = Z_x + \frac{16 - \lambda_f}{16 - 9}[\min(S_x, 1.5Z_x) - Z_x]}$$

For the slender range, the modulus is reduced in proportion to:

$$\boxed{Z_{e,x} = Z_x \frac{16}{\lambda_f}}$$

The exact branch limits in the FEAKalc implementation should be read together with these equations because the current code contains an additional flange slenderness branch beyond the nominal non-compact limit. This is worth retaining in the manual as an implementation detail rather than replacing it with a generic textbook classification.

Designer

3.13 PFC Section Slenderness

PFC sections use a separate design routine because the channel geometry is not doubly symmetric.

The PFC web slenderness is calculated as:

$$\boxed{\lambda_w = \frac{d - 2t_f}{t_w}\sqrt{\frac{f_{yw}}{250}}}$$

The flange slenderness differs from the I-section expression.

FEAKalc uses:

$$\boxed{\lambda_f = \frac{b_f - t_w}{t_f}\sqrt{\frac{f_{yf}}{250}}}$$

because the channel flange is treated as a single outstanding plate from the web rather than as two half-width flange outstands.

For compression, the implemented PFC limiting values are:

$$\boxed{\lambda_{w,\text{lim}} = 45}$$

and:

$$\boxed{\lambda_{f,\text{lim}} = 16}$$

The effective dimensions are therefore:

$$\boxed{d_e = (d - 2t_f)\frac{45}{\lambda_w}}$$

and:

$$\boxed{b_e = (b_f - t_w)\frac{16}{\lambda_f}}$$

subject in each case to the effective dimension not exceeding the actual dimension.

The PFC effective area is:

$$\boxed{A_e = A_g - [(d - 2t_f) - d_e]t_w - 2[(b_f - t_w) - b_e]t_f}$$

with:

$$\boxed{k_f = \frac{A_e}{A_g}}$$

and:

$$0 \leq k_f \leq 1$$

These equations are explicitly implemented in the PFC design routine.

Designer

3.14 PFC Major-Axis Classification

The PFC major-axis classification follows a similar web-versus-flange procedure.

The governing element is selected from:

$$\boxed{\frac{\lambda_w}{115} > \frac{\lambda_f}{16}}$$

For web-controlled behaviour:

$\lambda_w < 82 \quad \Rightarrow \quad$ Compact $82 \le \lambda_w < 115 \quad \Rightarrow \quad$ Non-Compact $\lambda_w \ge 115 \quad \Rightarrow \quad$ Slender

The resulting major-axis effective modulus follows:

$$Z_{e,x} = \min(S_x, 1.5Z_x)$$

for compact behaviour,

$$Z_{e,x} = Z_x + \frac{115 - \lambda_w}{115 - 82}[\min(S_x, 1.5Z_x) - Z_x]$$

for non-compact behaviour, and:

$$Z_{e,x} = Z_x \frac{115}{\lambda_w}$$

for slender behaviour.

For flange-controlled behaviour, the PFC routine uses the corresponding flange slenderness limits and interpolation procedure.

Designer

3.15 PFC Minor-Axis Bending

The PFC minor-axis calculation requires special treatment because a channel is not symmetric about its minor axis.

FEAKalc therefore calculates separate effective section properties for bending towards the two sides of the section:

$$\boxed{Z_{e,y,R}}$$

and:

$$\boxed{Z_{e,y,L}}$$

The corresponding section capacities are:

$$\boxed{M_{sy,R} = 0.9Z_{e,y,R}f_y}$$

and:

$$\boxed{M_{sy,L} = 0.9Z_{e,y,L}f_y}$$

The design engine subsequently calculates separate member bending capacities:

$$M_{by,R}$$

and:

$$M_{by,L}$$

and adopts the smaller value for the general minor-axis resistance:

$$\boxed{M_{by} = \min(M_{by,R}, M_{by,L})}$$

The FEAKalc design memo therefore identifies the PFC minor-axis classification as a **right/left checked** condition rather than assigning a single conventional minor-axis classification.

Designer

3.16 RHS and SHS Slenderness

Rectangular and square hollow sections are treated separately from open sections.

For these sections, the steel database supplies the overall depth d, width b, wall thickness t, section area and principal section properties. The current FEAKalc section lookup assigns:

$$\boxed{f_y = 350 \text{ MPa}}$$

to the RHS/SHS design input.

Designer

The flat web depth is represented by:

$$\boxed{d_w = d - 2t}$$

and the corresponding web slenderness takes the form:

$$\boxed{\lambda_w = \frac{d - 2t}{t}\sqrt{\frac{f_y}{250}}}$$

Similarly, for the transverse plate:

$$\boxed{\lambda_f = \frac{b - 2t}{t}\sqrt{\frac{f_y}{250}}}$$

The box-section routine uses these slenderness values to establish effective area, bending classification and shear resistance.

3.17 RHS/SHS Bending Classification

The box-section effective modulus routine uses the following web limits:

$$\boxed{\lambda_{wp} = 82}$$
$$\boxed{\lambda_{wn} = 115}$$

and flange limits:

$$\boxed{\lambda_{fp} = 30}$$
$$\boxed{\lambda_{fn} = 40}$$

The controlling plate is determined by comparing the relative slenderness of the web and flange.

For a web-controlled section:

$$\frac{\lambda_w}{115} > \frac{\lambda_f}{40}$$

the web governs the effective modulus.

For compact behaviour:

$$\boxed{Z_e = \min(S, 1.5Z)}$$

For non-compact behaviour:

$$\boxed{Z_e = Z + \frac{115 - \lambda_w}{115 - 82}[\min(S, 1.5Z) - Z]}$$

and for slender behaviour:

$$\boxed{Z_e = Z\frac{115}{\lambda_w}}$$

The corresponding flange-controlled expressions use the limits:

$$30 \quad \text{and} \quad 40$$

in place of 82 and 115. These limits and the interpolation procedure are implemented

3.18 CHS Sections

Circular hollow sections are handled by a dedicated design routine rather than being approximated as rectangular hollow sections.

The section database provides:

$$D, \quad t, \quad A, \quad I, \quad Z, \quad S, \quad r, \quad J$$

with equal principal properties:

$$\boxed{I_x = I_y}$$
$$\boxed{Z_x = Z_y}$$
$$\boxed{S_x = S_y}$$
$$\boxed{r_x = r_y}$$

The current FEAKalc section lookup assigns:

$$\boxed{f_y = 350 \text{ MPa}}$$

to CHS sections.

Because the circular section has identical properties about its principal bending axes, its treatment differs fundamentally from the plate-by-plate web/flange classification used for I-sections, PFCs and box sections.

The detailed CHS capacity equations are therefore better presented with the CHS member design section rather than forcing them into the web/flange classification framework.

3.19 Effect of Classification on Design Capacity

The classification calculation feeds directly into both compression and bending resistance.

For compression:

$$\boxed{A_g \to A_e \to k_f \to N_c}$$

where:

$$k_f = \frac{A_e}{A_g}$$

For bending:

$$\boxed{\lambda_w, \lambda_f \to \text{Section Classification} \to Z_e \to M_s \to M_b}$$

with:

$$\boxed{M_s = 0.9 Z_e f_y}$$

The section classification is therefore not merely descriptive output. It directly modifies the capacities used by the FEAKalc steel member design calculation.

The steel design memo reports the governing values of:

$$\lambda_w, \qquad \lambda_f, \qquad k_f$$

together with the major- and minor-axis section classifications, so that the basis of the calculated member resistance can be reviewed.

4 Compression Member Capacity and Flexural Buckling

4.1 General

A steel member subjected to axial compression may be governed either by the compressive resistance of its cross-section or by instability of the member about one of its principal axes.

FEAKalc therefore determines the compression capacity independently about the two principal buckling directions and adopts the smaller value:

$$\boxed{N_c = \min(N_{cx}, N_{cy})}$$

where:

- N_{cx} = compression capacity calculated using the effective length and radius of gyration associated with the x-axis;
- N_{cy} = corresponding compression capacity about the y-axis;
- N_c = governing member compression capacity.

The effective lengths used in these calculations are those determined from the three-dimensional restraint procedure described in Part 2. The design engine receives these as separate effective lengths for the two member directions.

4.2 Effective Section Area

Local plate slenderness is incorporated into the compression calculation through the section form factor:

$$\boxed{k_f = \frac{A_e}{A_g}}$$

where:

- A_e = effective cross-sectional area;
- A_g = gross cross-sectional area.

The value is limited by FEAKalc to:

$$\boxed{0 \leq k_f \leq 1}$$

For a fully effective section:

$$k_f = 1$$

and therefore:

$$A_e = A_g$$

For a section containing slender plate elements:

$$k_f < 1$$

and:

$$A_e = k_f A_g$$

The determination of A_e and k_f for the different section families was described in Part 3.

4.3 Nominal Member Slenderness

For each buckling direction, FEAKalc calculates the modified member slenderness:

$$\boxed{\lambda_n = \frac{L_e}{r}\sqrt{k_f}\sqrt{\frac{f_y}{250}}}$$

where:

- L_e = effective member length for the buckling direction, mm;
- r = radius of gyration about the corresponding axis, mm;
- k_f = section form factor;
- f_y = yield stress, MPa.

The conventional geometric slenderness is:

$$\frac{L_e}{r}$$

while the FEAKalc expression modifies this for both effective section area and steel yield strength:

$$\lambda_n = \left(\frac{L_e}{r}\right)\sqrt{k_f\frac{f_y}{250}}$$

This expression is used directly in the compression capacity routine.

4.4 Slenderness About the Two Principal Axes

For the x-axis:

$$\boxed{\lambda_{nx} = \frac{L_{e,x}}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250}}}$$

and for the y-axis:

$$\boxed{\lambda_{ny} = \frac{L_{e,y}}{r_y}\sqrt{k_f}\sqrt{\frac{f_y}{250}}}$$

Because both the effective lengths and radii of gyration may differ:

$$L_{e,x} \neq L_{e,y}$$

and:

$$r_x \neq r_y$$

it follows that generally:

$$\boxed{\lambda_{nx} \neq \lambda_{ny}}$$

The compression resistance is therefore calculated independently for each direction.

4.5 Section Constant α_a

From the modified member slenderness λ_n, FEAKalc calculates:

$$\boxed{\alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda_n^2 - 15.3\lambda_n + 2050}}$$

This parameter is then combined with the member section constant α_b to obtain the adjusted member slenderness.

4.6 Member Section Constant α_b

The implemented compression routine permits the member behaviour to be modified by the section constant:

$$\boxed{\alpha_b}$$

The adjusted slenderness is:

$$\boxed{\lambda = \lambda_n + \alpha_a \alpha_b}$$

Thus, the value of α_b directly affects the member compression reduction.

FEAKalc does not use one common α_b value for every section family. The value is selected according to the section type and, in some cases, whether the section is fully effective.

4.6.1 I-Sections

For the general I-section compression routine, FEAKalc initially adopts:

$$\boxed{\alpha_b = 0}$$

For a welded I-section having:

$$k_f < 1$$

the implementation changes this to:

$$\boxed{\alpha_b = 0.5}$$

Therefore:

$$\alpha_b = \begin{cases} 0.5 & \text{welded section with } k_f < 1 \\ 0 & \text{otherwise} \end{cases}$$

This logic is contained directly in the I-section compression capacity routine.

4.6.2 PFC Sections

PFC members use an explicitly assigned value based on the effective-area factor.

Where:

$$k_f \approx 1$$

FEAKalc adopts:

$$\boxed{\alpha_b = 0.5}$$

Where:

$$k_f < 1$$

the adopted value is:

$$\boxed{\alpha_b = 1.0}$$

Hence:

$$\boxed{\alpha_b = \begin{cases} 0.5 & k_f = 1 \\ 1.0 & k_f < 1 \end{cases}}$$

The same value is used for the compression calculations about both principal axes of the PFC.

4.6.3 RHS and SHS Sections

For rectangular and square hollow sections, the current implementation uses:

$$\boxed{\alpha_b = -0.5}$$

for the compression capacity calculation.

This value is passed explicitly to the compression capacity routine for the RHS/SHS design procedure.

4.6.4 CHS Sections

Circular hollow sections use:

$$\boxed{\alpha_b = \begin{cases} -0.5 & k_f < 1 \\ -1.0 & k_f = 1 \end{cases}}$$

The circular section is treated as having identical behaviour about its principal axes, and therefore only one compression capacity calculation is required in the current implementation.

4.7 Adjusted Member Slenderness

Once λ_n, α_a and α_b have been determined, the adjusted member slenderness is:

$$\boxed{\lambda = \lambda_n + \alpha_a \alpha_b}$$

This value is used for the subsequent compression reduction calculation.

The process may therefore be represented as:

$$\boxed{\frac{L_e}{r} \rightarrow \lambda_n \rightarrow \alpha_a \rightarrow \alpha_b \rightarrow \lambda}$$

4.8 Imperfection Parameter

FEAKalc next calculates the parameter:

$$\boxed{\eta = 0.00326(\lambda - 13.5)}$$

subject to:

$$\boxed{\eta \geq 0}$$

Therefore, where the calculated expression is negative:

$$0.00326(\lambda - 13.5) < 0$$

FEAKalc sets:

$$\boxed{\eta = 0}$$

This lower limit is explicitly imposed by the design routine.

4.9 Compression Reduction Parameter

The intermediate parameter ξ is then calculated as:

$$\boxed{\xi = \frac{\left(\frac{\lambda}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda}{90}\right)^2}}$$

The member compression reduction factor is subsequently:

$$\boxed{\alpha_c = \xi\left[1 - \sqrt{1 - \left(\frac{90}{\xi\lambda}\right)^2}\right]}$$

The complete reduction process implemented by FEAKalc is therefore:

$$\boxed{\lambda_n \to \alpha_a \to \lambda \to \eta \to \xi \to \alpha_c}$$

These equations are implemented directly in both compression-capacity helper routines used by the steel design engine.

4.10 Section Compression Capacity

Before member buckling is considered, FEAKalc calculates the effective section compression capacity as:

$$\boxed{N_s = 0.9\, k_f A_g f_y}$$

Since:

$$A_e = k_f A_g$$

this may also be written as:

$$\boxed{N_s = 0.9 A_e f_y}$$

where:

- A_g = gross area in mm^2;
- A_e = effective area in mm^2;
- f_y = yield stress in MPa;

- N_s = section compression capacity in N.

The factor:

$$\boxed{\phi = 0.9}$$

is therefore already incorporated into the value returned by the FEAKalc compression-capacity routine.

4.11 Member Compression Capacity

The member compression capacity is obtained by applying the compression reduction factor:

$$\boxed{N_c = \alpha_c N_s}$$

or:

$$\boxed{N_c = 0.9\alpha_c k_f A_g f_y}$$

FEAKalc additionally limits the member compression capacity so that:

$$\boxed{N_c \leq N_s}$$

Hence, the implemented relationship is effectively:

$$\boxed{N_c = \min[\alpha_c N_s,\ N_s]}$$

This prevents the calculated member capacity from exceeding the effective section compression capacity.

4.12 Compression Capacity About Each Axis

The complete calculation is performed independently about the principal axes.

For the x-axis:

$$\lambda_{nx} = \frac{L_{e,x}}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250}}$$

which ultimately gives:

$$\boxed{N_{cx} = 0.9\alpha_{cx} k_f A_g f_y}$$

subject to:

$$N_{cx} \leq N_s$$

For the y-axis:

$$\lambda_{ny} = \frac{L_{e,y}}{r_y}\sqrt{k_f}\sqrt{\frac{f_y}{250}}$$

giving:

$$\boxed{N_{cy} = 0.9\alpha_{cy} k_f A_g f_y}$$

subject to:

$$N_{cy} \le N_s$$

The governing compression capacity is:

$$\boxed{N_c = \min(N_{cx}, N_{cy})}$$

The FEAKalc design memo reports all three values:

$$\boxed{N_{cx}, \qquad N_{cy}, \qquad N_c}$$

so that the governing buckling direction can be identified.

4.13 Relationship to FEAKalc Effective Lengths

The client-side model determines separate effective lengths for the local 22 and 33 restraint directions and transfers these to the server-side steel design calculation.

Conceptually:

$$L_{e,22} \rightarrow \lambda_{n,22} \rightarrow N_{c,22}$$

and:

$$L_{e,33} \rightarrow \lambda_{n,33} \rightarrow N_{c,33}$$

with:

$$\boxed{N_c = \min\left(N_{c,22}, N_{c,33}\right)}$$

Accordingly, increasing the effective length in one direction increases the member slenderness:

$$L_e \uparrow \quad \Rightarrow \quad \lambda_n \uparrow$$

which generally reduces:

$$\alpha_c$$

and consequently reduces:

$$N_c$$

The effective-length determination discussed in Part 2 therefore feeds directly into the compression resistance calculation.

4.14 Minimum Effective Length Used by the Design Engine

There is an important implementation detail in the current steel design engine.

Before calculating the capacities of I-sections, PFCs and RHS/SHS sections, FEAKalc imposes:

$$\boxed{L_{e,x} \ge 2000 \text{ mm}}$$

and:

$$\boxed{L_{e,y} \ge 2000 \text{ mm}}$$

For the LTB length, where a positive user-defined value is supplied:

$$\boxed{L_{LTB} \ge 2000 \text{ mm}}$$

For CHS members, the current routine similarly imposes a minimum L_x of:

$$\boxed{2000 \text{ mm}}$$

before the compression calculation.

4.15 I-Section Compression

For I-sections, the effective-area factor k_f is obtained from the web and flange slenderness calculation described in Part 3.

The compression calculation then proceeds as:

$$\boxed{\lambda_{nx} = \frac{L_{e,x}}{r_x}\sqrt{k_f}\sqrt{\frac{f_y}{250}}}$$

$$\boxed{\lambda_{ny} = \frac{L_{e,y}}{r_y}\sqrt{k_f}\sqrt{\frac{f_y}{250}}}$$

with the I-section α_b rule applied.

The resulting capacities are:

$$N_{cx}$$

and:

$$N_{cy}$$

and the governing resistance is:

$$\boxed{N_c = \min(N_{cx}, N_{cy})}$$

Thus, even where the cross-section itself is fully effective, the member may still be governed by flexural buckling about its weaker principal direction.

4.16 PFC Compression

PFC members use the same general compression reduction equations, but with the PFC-specific value of α_b.

The program determines:

$$\alpha_b = \begin{cases} 0.5 & k_f = 1 \\ 1.0 & k_f < 1 \end{cases}$$

and calculates:

$$\boxed{N_{cx} = N_c(A_g, r_x, f_y, k_f, L_{e,x}, \alpha_b)} \boxed{N_{cy} = N_c(A_g, r_y, f_y, k_f, L_{e,y}, \alpha_b)}$$

The governing PFC compression capacity is:

$$\boxed{N_c = \min(N_{cx}, N_{cy})}$$

This calculation is explicitly performed about both axes in the PFC design routine.

4.17 RHS and SHS Compression

For RHS and SHS sections, FEAKalc adopts:

$$\boxed{\alpha_b = -0.5}$$

The capacities are therefore obtained from:

$$\boxed{N_{cx} = N_c\left(A_g, r_x, f_y, k_f, L_{e,x}, -0.5\right)}$$

and:

$$\boxed{N_{cy} = N_c\left(A_g, r_y, f_y, k_f, L_{e,y}, -0.5\right)}$$

with:

$$\boxed{N_c = \min(N_{cx}, N_{cy})}$$

The effective-area factor k_f used here is derived from the box-section plate slenderness calculation described in Part 3.

4.18 CHS Compression

Circular hollow sections are treated differently because the section properties are identical about the principal axes:

$$r_x = r_y$$

The current implementation calculates one compression capacity using r_x and the supplied L_x:

$$\boxed{N_c = N_c\left(A_g, r_x, f_y, k_f, L_x, \alpha_b\right)}$$

and assigns:

$$\boxed{N_{cx} = N_{cy} = N_c}$$

The adopted section constant is:

$$\alpha_b = \begin{cases} -0.5 & k_f < 1 \\ -1.0 & k_f = 1 \end{cases}$$

This equality of N_{cx} and N_{cy} is therefore an explicit feature of the present CHS implementation.

4.19 Axial Compression Utilisation

Once the governing compression capacity has been established, FEAKalc calculates the axial utilisation as:

$$\boxed{U_N = \frac{|N^*|}{N_c}}$$

where N_c already includes the 0.9 capacity factor used by the compression routine.

The axial compression check therefore satisfies the design requirement when:

$$\boxed{\frac{|N^*|}{N_c} \leq 1.0}$$

or equivalently:

$$\boxed{|N^*| \le N_c}$$

The absolute value of the analysis axial action is used in the current implementation.

4.20 Compression Design Sequence

The complete FEAKalc compression calculation may be summarised as:

$$\boxed{\text{Section Geometry} \to \lambda_{\text{plate}} \to A_e \to k_f}$$

followed by:

$$\boxed{\text{3D Member Restraint} \to L_e}$$

and:

$\boxed{L_e,\ r,\ k_f,\ f_y \to \lambda_n}$ $\boxed{\lambda_n \to \alpha_a \to \lambda \to \eta \to \xi \to \alpha_c}$

Finally:

$\boxed{N_s = 0.9 k_f A_g f_y}$ $\boxed{N_c = \min(\alpha_c N_s, N_s)}$

with separate calculations about the two principal directions:

$$\boxed{N_{cx}, \quad N_{cy}}$$

and:

$$\boxed{N_c = \min(N_{cx}, N_{cy})}$$

The design check is then:

$$\boxed{U_N = \frac{|N^*|}{N_c} \le 1.0}$$

This is the compression-capacity sequence implemented by the current FEAKalc steel design engine.

5 Bending Capacity and Lateral-Torsional Buckling

5.1 General

The bending resistance of a steel member depends on both the capacity of the cross-section and the stability of the member between points of restraint.

FEAKalc therefore distinguishes between:

$$\boxed{M_s}$$

the section bending capacity, and:

$$\boxed{M_b}$$

the member bending capacity after allowance for lateral-torsional buckling.

The general sequence is:

$$\boxed{\text{Section Slenderness} \rightarrow Z_e \rightarrow M_s \rightarrow M_o \rightarrow \alpha_s \rightarrow M_b}$$

The effective section modulus Z_e is obtained from the section classification procedures described in Part 3.

5.2 Section Bending Capacity

For a given bending axis, FEAKalc calculates the section bending capacity as:

$$\boxed{M_s = 0.9\, Z_e f_y}$$

where:

- Z_e = effective section modulus;
- f_y = steel yield stress;
- 0.9 = capacity reduction factor incorporated in the implementation.

For major-axis bending:

$$\boxed{M_{sx} = 0.9 Z_{e,x} f_y}$$

and for minor-axis bending:

$$\boxed{M_{sy} = 0.9 Z_{e,y} f_y}$$

The resulting capacities are calculated initially in:

Nmm

and subsequently converted to:

kNm

for reporting.

The current FEAKalc steel engine uses this relationship for I-sections, PFCs and hollow sections, with the appropriate effective modulus for the section family.

5.3 Effective Section Modulus

The effective modulus used in the bending calculation depends on the section classification.

For a compact section:

$$\boxed{Z_e = \min(S, 1.5Z)}$$

For a non-compact section:

$$\boxed{Z_e = Z + \frac{\lambda_n - \lambda}{\lambda_n - \lambda_p}[\min(S, 1.5Z) - Z]}$$

For a slender section:

$$\boxed{Z_e = Z\frac{\lambda_n}{\lambda}}$$

where:

- Z = elastic section modulus;
- S = plastic section modulus;
- λ = calculated plate slenderness;
- λ_p = compact limit;
- λ_n = non-compact limit.

Thus:

$$\boxed{Z_e \leq \min(S, 1.5Z)}$$

The section bending capacity therefore reduces as plate slenderness increases.

5.4 Major-Axis Bending

For I-sections and PFC sections, the major-axis bending action corresponds to:

$$\boxed{M_{33}^*}$$

The section capacity is:

$$\boxed{M_{sx} = 0.9Z_{e,x}f_y}$$

However, for an open steel section this capacity may not be fully available if the compression flange is insufficiently restrained against lateral movement and twisting.

FEAKalc therefore applies a member bending capacity calculation to determine:

$$\boxed{M_{bx}}$$

which is used in the major-axis utilisation check.

The design requirement is:

$$\boxed{U_{M33} = \frac{|M_{33}^*|}{M_{bx}} \leq 1.0}$$

5.5 Lateral-Torsional Buckling Length

The lateral-torsional buckling calculation uses a separate effective length:

$$\boxed{L_{LTB}}$$

This is distinct from the axial buckling lengths:

$$L_{e,22}$$

and:

$$L_{e,33}$$

Where an explicit LTB length is assigned by the user:

$$\boxed{L = L_{LTB}}$$

is used in the major-axis member bending calculation.

Where no positive LTB length has been supplied, the current implementation reverts to the effective length used for the major-axis member calculation.

For the I-section routine, the fallback relationship is:

$$\boxed{L = L_{e,x}}$$

when:

$$L_{LTB} \leq 0$$

The PFC and RHS/SHS routines follow the same general logic.

5.6 Minimum LTB Length

Before the member bending calculation is performed, the current design implementation imposes:

$$\boxed{L_{LTB} \geq 2000 \text{ mm}}$$

where a positive LTB length has been specified.

Hence:

$$0 < L_{LTB} < 2000 \text{ mm}$$

is replaced by:

$$\boxed{L_{LTB} = 2000 \text{ mm}}$$

This is a FEAKalc implementation rule and should not be interpreted as a general theoretical requirement independent of the software.

5.7 Elastic Buckling Moment

The FEAKalc member bending routine calculates an elastic buckling-type moment from the minor-axis flexural stiffness, torsional stiffness and warping stiffness.

The implemented equation is:

$$\boxed{M_o = \sqrt{\left(\frac{\pi^2 E I_y}{L^2}\right)\left[GJ + \frac{\pi^2 E I_w}{L^2}\right]}}$$

where:

- E = Young's modulus of steel;
- G = shear modulus;
- I_y = minor-axis second moment of area;
- J = torsional constant;
- I_w = warping constant;
- L = lateral-torsional buckling length.

The constants used by FEAKalc are:

$$\boxed{E = 200000 \text{ MPa}}$$

and:

$$\boxed{G = 80000 \text{ MPa}}$$

The expression implemented in the steel design engine is therefore:

$$\boxed{M_o = \sqrt{\left(\frac{\pi^2 (200000) I_y}{L^2}\right)\left[80000J + \frac{\pi^2 (200000) I_w}{L^2}\right]}}$$

This is the principal stability term used in the current FEAKalc lateral-torsional buckling calculation.

5.8 Relative Bending Slenderness

The section bending capacity and elastic buckling moment are combined through the ratio:

$$\boxed{\frac{M_s/\phi}{M_o}}$$

Since the FEAKalc section capacity already contains:

$$\phi = 0.9$$

the implementation uses:

$$\boxed{\frac{M_s}{0.9M_o}}$$

as the principal relative bending parameter.

Let:

$$\boxed{\lambda_m = \frac{M_s}{0.9M_o}}$$

Then increasing the unrestrained length reduces M_o, which increases λ_m:

$$L \uparrow \Rightarrow M_o \downarrow \Rightarrow \lambda_m \uparrow$$

and therefore reduces the available member bending capacity.

5.9 Slenderness Reduction Factor

FEAKalc calculates the member bending reduction factor:

$$\boxed{\alpha_s = 0.6\left[\sqrt{\left(\frac{M_s}{0.9M_o}\right)^2 + 3} - \frac{M_s}{0.9M_o}\right]}$$

or, using λ_m:

$$\boxed{\alpha_s = 0.6\left[\sqrt{\lambda_m^2 + 3} - \lambda_m\right]}$$

This reduction factor represents the influence of lateral-torsional instability on the available member bending resistance.

5.10 Moment Modification Factor

The current member bending routine adopts:

$$\boxed{\alpha_m = 1.0}$$

The member bending capacity is then calculated as:

$$\boxed{M_b = \alpha_m \alpha_s M_s}$$

Since:

$$\alpha_m = 1$$

the implemented relationship reduces to:

$$\boxed{M_b = \alpha_s M_s}$$

The code additionally limits the result such that:

$$\boxed{M_b \leq M_s}$$

Accordingly, the effective implementation is:

$$\boxed{M_b = \min(\alpha_s M_s,\ M_s)}$$

This ensures that the member bending capacity cannot exceed the section bending capacity.

5.11 Complete Lateral-Torsional Buckling Calculation

The complete FEAKalc calculation may therefore be written as:

$$\boxed{M_s = 0.9 Z_e f_y}$$

followed by:

$$\boxed{M_o = \sqrt{\frac{\pi^2 E I_y}{L^2}\left(GJ + \frac{\pi^2 E I_w}{L^2}\right)}}$$

Then:

$$\boxed{\lambda_m = \frac{M_s}{0.9M_o}}$$

and:

$$\boxed{\alpha_s = 0.6\left(\sqrt{\lambda_m^2 + 3} - \lambda_m\right)}$$

Finally:

$$\boxed{M_b = \min(\alpha_s M_s,\ M_s)}$$

The member major-axis utilisation is:

$$\boxed{U_{M33} = \frac{|M_{33}^*|}{M_b}}$$

5.12 Effect of Unrestrained Length

The lateral-torsional buckling equation illustrates the importance of restraint length.

The elastic buckling moment contains:

$$\frac{1}{L^2}$$

in both the flexural and warping terms.

Thus, approximately:

$$\boxed{L \uparrow \Rightarrow M_o \downarrow}$$

which results in:

$$\boxed{\alpha_s \downarrow}$$

and hence:

$$\boxed{M_b \downarrow}$$

Conversely:

$$\boxed{L \downarrow \Rightarrow M_o \uparrow \Rightarrow M_b \uparrow}$$

until the section capacity becomes governing:

$$\boxed{M_b = M_s}$$

Accordingly, providing intermediate lateral restraint may substantially increase the major-axis bending resistance of an open steel section.

5.13 Major-Axis Bending of I-Sections

For an I-section:

$$\boxed{M_{sx} = 0.9Z_{e,x}f_y}$$

The member capacity is calculated using:

$$I_y, \quad J, \quad I_w$$

and the LTB length.

Where:

$$L_{LTB} > 0$$

FEAKalc uses:

$$\boxed{M_{bx} = M_b(M_{sx}, I_y, J, I_w, L_{LTB})}$$

Otherwise:

$$\boxed{M_{bx} = M_b(M_{sx}, I_y, J, I_w, L_x)}$$

The design utilisation is:

$$\boxed{U_{M33} = \frac{|M_{33}^*|}{M_{bx}}}$$

This logic is explicitly implemented in the I-section design function.

5.14 Minor-Axis Bending of I-Sections

For minor-axis bending:

$$\boxed{M_{sy} = 0.9 Z_{e,y} f_y}$$

The current FEAKalc implementation subsequently passes this capacity through the same member bending function:

$$\boxed{M_{by} = M_b(M_{sy}, I_y, J, I_w, L_y)}$$

The minor-axis utilisation is therefore:

$$\boxed{U_{M22} = \frac{|M_{22}^*|}{M_{by}}}$$

This is the calculation currently performed by the code and should be documented as the FEAKalc implementation.

5.15 PFC Major-Axis Bending

For a PFC section, the major-axis section capacity is:

$$\boxed{M_{sx} = 0.9 Z_{e,x} f_y}$$

The member bending capacity is calculated from:

$$\boxed{M_{bx} = M_b(M_{sx}, I_y, J, I_w, L)}$$

where:

$$L = \begin{cases} L_{LTB} & L_{LTB} > 0 \\ L_x & L_{LTB} \le 0 \end{cases}$$

The major-axis utilisation is then:

$$\boxed{U_{M33} = \frac{|M_{33}^*|}{M_{bx}}}$$

The PFC design routine therefore uses the same general lateral-torsional buckling calculation as the I-section routine.

5.16 PFC Minor-Axis Bending

Because a PFC is not symmetric about the minor axis, separate effective section moduli are determined for bending in the two directions:

$$Z_{e,y,R}$$

and:

$$Z_{e,y,L}$$

The corresponding section capacities are:

$$\boxed{M_{sy,R} = 0.9Z_{e,y,R}f_y}$$

and:

$$\boxed{M_{sy,L} = 0.9Z_{e,y,L}f_y}$$

FEAKalc then calculates:

$$\boxed{M_{by,R} = M_b(M_{sy,R}, I_y, J, I_w, L_y)}$$

and:

$$\boxed{M_{by,L} = M_b(M_{sy,L}, I_y, J, I_w, L_y)}$$

The general minor-axis capacity adopted is:

$$\boxed{M_{by} = \min\left(M_{by,R}, M_{by,L}\right)}$$

and therefore:

$$\boxed{U_{M22} = \frac{|M_{22}^*|}{M_{by}}}$$

This ensures that the less favourable side of the PFC governs the general minor-axis capacity.

5.17 RHS and SHS Bending

For RHS and SHS sections, the effective section moduli are determined independently about the principal axes:

$$Z_{e,x}$$

and:

$$Z_{e,y}$$

The corresponding section capacities are:

$$\boxed{M_{sx} = 0.9Z_{e,x}f_y}$$

and:

$$\boxed{M_{sy} = 0.9 Z_{e,y} f_y}$$

The current FEAKalc routine calculates member capacities using:

$$\boxed{M_{bx} = M_b(M_{sx}, I_y, J, I_w, L_x)}$$

or the supplied LTB length where applicable, and:

$$\boxed{M_{by} = M_b(M_{sy}, I_y, J, I_w, L_y)}$$

The resulting utilisation checks are:

$$\boxed{U_{M33} = \frac{|M_{33}^*|}{M_{bx}}}$$

and:

$$\boxed{U_{M22} = \frac{|M_{22}^*|}{M_{by}}}$$

For the current RHS/SHS database input:

$$I_w = 0$$

so the warping term in the implemented member bending equation becomes zero.

5.18 CHS Bending

For circular hollow sections, the principal section properties are equal:

$$\boxed{Z_x = Z_y}$$

and:

$$\boxed{S_x = S_y}$$

The current CHS routine therefore calculates one effective modulus:

$$Z_e$$

and one section bending capacity:

$$\boxed{M_s = 0.9 Z_e f_y}$$

The member capacity is:

$$\boxed{M_b = M_b(M_s, I_y, J, I_w, L_x)}$$

and FEAKalc assigns:

$$\boxed{M_{bx} = M_{by} = M_b}$$

Hence:

$$\boxed{U_{M33} = \frac{|M_{33}^*|}{M_b}}$$

and:

$$\boxed{U_{M22} = \frac{|M_{22}^{*}|}{M_b}}$$

The current CHS database input uses:

$$\boxed{I_w = 0}$$

so the member bending stability expression depends on the flexural and St Venant torsional terms retained by the implemented function.

5.19 Section Capacity Versus Member Capacity

An important distinction in the FEAKalc result is:

$$\boxed{M_s = \text{section capacity}}$$

whereas:

$$\boxed{M_b = \text{member capacity}}$$

with:

$$\boxed{M_b \leq M_s}$$

For a well-restrained member:

$$M_o \gg M_s$$

and therefore the lateral-torsional buckling reduction becomes small, so approximately:

$$\boxed{M_b \to M_s}$$

For a long unrestrained member:

$$M_o \downarrow$$

and:

$$\boxed{M_b < M_s}$$

Thus a section may have adequate cross-sectional bending strength but still fail the member bending check because of instability.

5.20 Bending Design Sequence

The complete FEAKalc bending design sequence may be summarised as:

$$\boxed{\lambda_w, \lambda_f \to \text{classification} \to Z_e}$$

then:

$$\boxed{M_s = 0.9 Z_e f_y}$$

followed by:

$$\boxed{L_{LTB}, I_y, J, I_w, E, G \to M_o}$$

then:

$$\boxed{M_s, M_o \to \alpha_s}$$

and:

$$\boxed{M_b = \min(\alpha_s M_s, M_s)}$$

Finally:

$$\boxed{U_{M33} = \frac{|M_{33}^*|}{M_{bx}}}$$

and:

$$\boxed{U_{M22} = \frac{|M_{22}^*|}{M_{by}}}$$

The values:

$$M_{sx}, \quad M_{sy}, \quad M_{bx}, \quad M_{by}$$

are retained in the FEAKalc station results and are available to the design memo.

6 Shear Capacity

6.1 General

FEAKalc checks shear resistance independently at each analysis station using the local member shear actions V2* and V3*. The current steel design engine calculates a design shear capacity for each supported section family and compares the absolute analysis shear actions with that capacity.

$$U\_V2 = |V2^*| \,/\, V\_v$$

$$U\_V3 = |V3^*| \,/\, V\_v$$

where V_v is the design shear capacity returned by the applicable section routine. For I-sections and PFCs the present calculation is web-based. RHS and SHS sections are evaluated in both principal wall directions before a governing value is adopted, while CHS sections use a section-area-based expression.

6.2 Basic Shear Yield Relationship

For open sections, the basic shear resistance used by the implementation is based on the shear yield stress:

$$\tau\_y = 0.6\, f\_y$$

With the capacity reduction factor incorporated in the calculation:

$$\varphi = 0.9$$

the design shear resistance for an effective shear area A_v is:

$$V\_v = 0.9\,(0.6\, f\_y\, A\_v) = 0.54\, f\_y\, A\_v$$

Section dimensions are supplied in millimetres and yield stress in MPa. The resulting force in newtons is divided by 1000 for reporting in kN.

6.3 I-Section Shear Capacity

For I-sections, FEAKalc bases the shear resistance on the web. The effective web depth used by the current routine depends on whether the section is rolled or welded.

$$d\_v = d \qquad \textit{rolled I-section}$$

$$d\_v = d - 2\, t\_f \qquad \textit{welded I-section}$$

The unreduced design web shear capacity is therefore:

$$V\_v0 = 0.9\ (0.6\ f\_yw\ d\_v\ t\_w)$$

or, in kN:

$$V\_v0 = 0.54\ f\_yw\ d\_v\ t\_w\ /\ 1000$$

The web slenderness used for the shear reduction is the same yield-stress-modified web slenderness introduced in Part 3:

$$\lambda\_w = ((d - 2\ t\_f)\ /\ t\_w)\ \sqrt{(f\_yw\ /\ 250)}$$

Where the web slenderness does not exceed 82, the full web shear capacity is retained. Where λ_w exceeds 82, FEAKalc reduces the capacity by the square of the slenderness ratio:

$$V\_v = V\_v0 \qquad \text{for } \lambda\_w \leq 82$$

$$V\_v = V\_v0\ (82\ /\ \lambda\_w)^2 \qquad \text{for } \lambda\_w > 82$$

The resulting value is stored as the web shear capacity and is presently used for both local shear directions in the I-section station check.

6.4 PFC Shear Capacity

PFC sections use the same general web-shear concept, with the overall channel depth used in the present implementation. The unreduced capacity is:

$$V\_v0 = 0.9\ (0.6\ f\_yw\ d\ t\_w)$$

$$V\_v0 = 0.54\ f\_yw\ d\ t\_w\ /\ 1000 \qquad kN$$

The same slender-web reduction is applied:

$$V\_v = V\_v0 \qquad \text{for } \lambda\_w \leq 82$$

$$V\_v = V\_v0\ (82\ /\ \lambda\_w)^2 \qquad \text{for } \lambda\_w > 82$$

As for I-sections, this calculated web capacity is currently used for both V2* and V3* utilisation checks.

6.5 RHS and SHS Shear Capacity

Rectangular and square hollow sections are treated separately because two opposite walls participate in resisting shear. FEAKalc calculates a strong-direction and a weak-direction shear resistance, including geometry-dependent box factors.

For the strong direction, the box factor is:

$$k\_vx = 2\ /\ [1.5\ ((2\ b + d)\ /\ (3\ b + d)) + 0.9]$$

$$k\_vx \leq 1.0$$

The corresponding strong-direction design shear capacity is:

$$V\_vx = 0.9\ (0.6\ f\_y\ (d - 2\ t\_f)\ t\_w)\ (2\ k\_vx)\ /\ 1000$$

Where λ_w > 82, this capacity is reduced by:

$$V\_vx \rightarrow V\_vx\ (82\ /\ \lambda\_w)^2$$

For the orthogonal direction, the box factor is:

$$k\_vy = 2\ /\ [1.5\ ((2\ d + b)\ /\ (3\ d + b)) + 0.9]$$

$$k\_vy \leq 1.0$$

and the weak-direction design shear capacity is:

$$V\_vy = 0.9\ (0.6\ f\_y\ (b - 2\ t\_w)\ t\_f)\ (2\ k\_vy) / 1000$$

Where λ_f > 82, the corresponding slenderness reduction is:

$$V\_vy \rightarrow V\_vy\ (82 / \lambda\_f)^2$$

The current box-section routine then adopts the lesser value as the general shear capacity passed to the station finalisation calculation:

$$V\_v = min(V\_vx,\ V\_vy)$$

Consequently, the current result finalisation uses the lesser RHS/SHS directional shear resistance for both local shear utilisation checks.

6.6 CHS Shear Capacity

Circular hollow sections use a separate section-area-based expression. The present FEAKalc CHS design routine calculates:

$$V\_v = 0.324\ f\_y\ A\_g / 1000$$

where A_g is the gross cross-sectional area in mm^2 and f_y is in MPa. Because the circular section is rotationally symmetric, the same capacity is used for the two principal local shear directions.

6.7 Shear Utilisation and Acceptance

At every design station, the shear utilisation ratios are:

$$U\_V2 = |V2^*| / V\_v$$

$$U\_V3 = |V3^*| / V\_v$$

The individual shear requirement is satisfied when:

$$U\_V2 \leq 1.0 \quad and \quad U\_V3 \leq 1.0$$

Equivalently:

$$|V2^*| \leq V\_v \quad and \quad |V3^*| \leq V\_v$$

The two shear utilisations are retained with the other station checks and may govern the reported member utilisation.

6.8 Current Scope of the Shear Check

The present FEAKalc steel design implementation evaluates shear directly against the calculated section shear capacity. The supplied design engine does not currently include a separate high-shear reduction of bending capacity or a combined shear-bending interaction expression. Torsion T* is transferred to the station result and reported, but it is not presently incorporated into the shear capacity or interaction utilisation.

Accordingly, the implemented shear design sequence is:

$$Section\ geometry + f\_y \rightarrow A\_v \rightarrow slenderness\ reduction \rightarrow V\_v \rightarrow U\_V2,\ U\_V3$$

7 Combined Actions and Governing Design

7.1 General

In addition to the individual axial, bending and shear checks, FEAKalc evaluates a combined axial-force and biaxial-bending utilisation at every design station. The purpose of the interaction check is to identify members for which the simultaneous presence of axial force and bending about both principal axes is more critical than any individual action considered alone.

7.2 Individual Utilisation Components

The station interaction is assembled from the same component utilisation ratios used in the individual checks:

$$U\_N = |N^*| / N\_c$$

$$U\_M33 = |M33^*| / M\_bx$$

$$U\_M22 = |M22^*| / M\_by$$

where N_c is the governing member compression capacity, M_bx is the major-axis member bending capacity and M_by is the minor-axis member bending capacity. These capacities already include the reductions and capacity factors described in Parts 3 to 5.

7.3 Implemented Axial-Biaxial Bending Interaction

The current FEAKalc steel design engine uses a linear utilisation summation for the combined axial and biaxial bending check:

$$U\_I = U\_N + U\_M33 + U\_M22$$

Substituting the component ratios gives:

$$U\_I = |N^*|/N\_c + |M33^*|/M\_bx + |M22^*|/M\_by$$

The combined action requirement is satisfied when:

$$U\_I \leq 1.0$$

This equation documents the formulation actually implemented in the current FEAKalc design engine. It should therefore be interpreted as the current program interaction check rather than as a substitute for every clause-specific interaction expression that may be applicable to a particular structural situation.

7.4 Treatment of Axial Action

The current interaction routine uses the absolute value of the station axial action:

$$|N^*|$$

and compares this with the member compression capacity N_c. The same axial utilisation is used in the independent axial check and in the combined interaction check. The present routine therefore does not introduce a separate tension-capacity branch within this steel interaction calculation.

7.5 Station Governing Check

At each station, FEAKalc evaluates the individual and combined utilisations:

$$U\_N,\ U\_M33,\ U\_M22,\ U\_V2,\ U\_V3,\ U\_I$$

The governing utilisation for station j is the maximum of these values:

$$U\_j = max(U\_N, U\_M33, U\_M22, U\_V2, U\_V3, U\_I)$$

The corresponding check name is retained as the governing check for that station. Typical reported governing checks include axial, M33 major bending, M22 minor bending, V2 shear, V3 shear and N + M33 + M22 interaction.

7.6 Governing Station for a Member

A frame member is checked at all available FEAKalc analysis stations. For a particular load combination, the governing member utilisation is:

$$U\_member,LC = max(U\_1, U\_2, \ldots, U\_m)$$

where m is the number of design stations. The station producing this maximum is retained together with its station name and distance S along the member.

7.7 Load Combination Envelope

The client-side steel design workflow repeats the complete station-by-station calculation for each FEAKalc load combination identified by the LC- prefix. For a member checked under q applicable combinations, the final reported utilisation is:

$$U\_member = max(U\_member,1, U\_member,2, \ldots, U\_member,q)$$

The load combination producing this maximum is stored as the governing load case. The final displayed result is therefore an envelope over both analysis stations and applicable design load combinations.

7.8 PASS/FAIL Classification

The design status is assigned directly from the governing utilisation. A station, and ultimately the member, is reported as passing when:

$$U \leq 1.0$$

and failing when:

$$U > 1.0$$

The reported utilisation is dimensionless and is intended to provide a direct demand-to-capacity indication. A value approaching 1.0 indicates that the governing design resistance is being approached.

7.9 Combined Design Sequence

The complete governing-design process may be summarised as:

$$\text{Analysis actions} \rightarrow \text{section/member capacities} \rightarrow \text{individual utilisations} \rightarrow \text{interaction utilisation}$$
$$\rightarrow \text{governing station} \rightarrow \text{governing load combination} \rightarrow \text{final member utilisation and status}$$

8 Design Scope, Interpretation and Closing

8.1 Scope of the Current Steel Design Module

The FEAKalc 3D steel design module provides an integrated member-design workflow linking finite element analysis, section properties, effective lengths, section classification, member stability and design utilisation. The current implementation supports I-sections, PFCs, RHS/SHS and CHS sections and performs station-by-station checks for axial action, biaxial bending, shear and combined axial force with biaxial bending.

The design procedures documented in this manual describe the calculations implemented in the current software version. Where the program contains a specific implementation rule, such as a minimum effective length or a particular interaction formulation, that rule has been identified as an FEAKalc implementation detail.

8.2 Design Inputs and Engineering Judgement

The reliability of a member design depends on the analysis model, assigned section, material properties, restraint assumptions and load combinations. Automatic effective-length determination is based on the geometry and connectivity represented in the FEAKalc model. Users may override effective lengths or lateral-torsional buckling lengths where the physical restraint condition is known to differ from the automatically inferred model behaviour.

The design output should therefore be reviewed together with the structural model and the actual restraint and connection conditions. Program-calculated utilisation is not a substitute for engineering judgement regarding load paths, connection behaviour, local detailing, construction sequence or restraints that are not represented by the analysis model.

8.3 Current Implementation Boundaries

The current steel design engine has defined boundaries that are relevant when interpreting the results:

- torsional action T* is transferred and reported but is not presently included as a separate capacity or interaction check;
- the combined axial and biaxial bending check currently uses the linear utilisation expression U_I = U_N + U_M33 + U_M22;
- the present shear routines do not include a separate high-shear reduction of bending capacity or shear-bending interaction;
- the current I-section and PFC routines use a common calculated web shear capacity for both local shear utilisation checks;
- the current RHS/SHS routine adopts the lesser calculated directional shear capacity as the general shear capacity used in station finalisation; and
- the current member bending routine adopts a moment modification factor α_m = 1.0.

These items describe the present implementation and are included so that the reported design results can be interpreted transparently.

8.4 Result Traceability

For each steel member, FEAKalc retains the governing load combination, governing station, governing check, individual utilisation ratios and principal calculated capacities. This allows the overall member result to be traced back to the station and action combination that controls the design.

The steel design memo additionally reports member length, effective lengths, lateral-torsional buckling length where specified, section classification parameters, compression and bending capacities, applied actions at stations and station-by-station utilisation results.

8.5 Closing Summary

The FEAKalc 3D steel design workflow can be represented by the following sequence:

3D finite element analysis

↓

Design load-combination actions at member stations

↓

Section classification and effective section properties

↓

Effective length and member stability assessment

↓

Compression, bending and shear capacities

↓

Combined action and station utilisation checks

↓

Load-combination envelope and governing member result

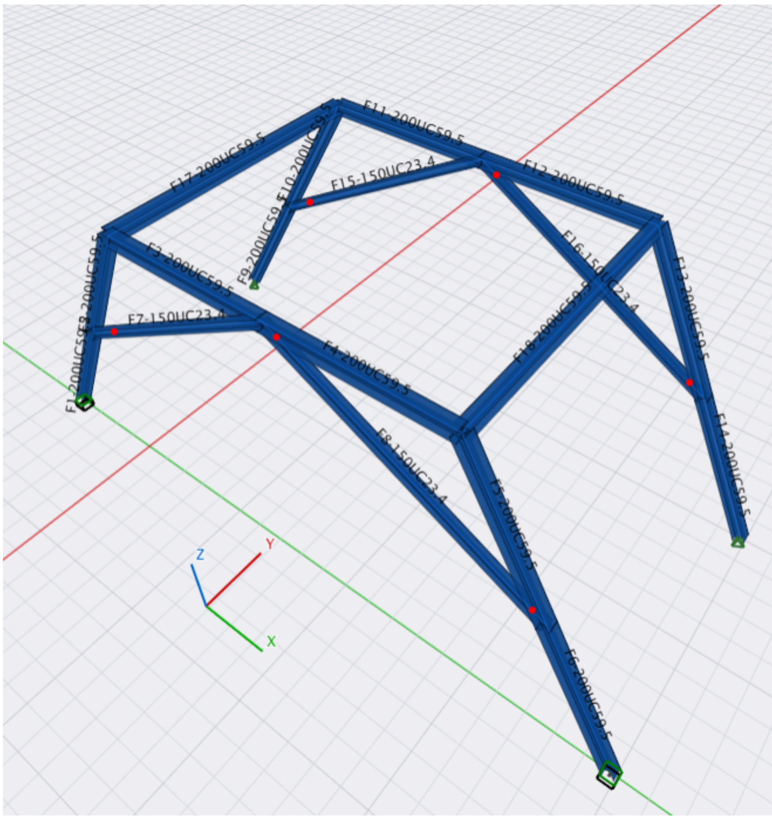
This provides a direct link between the three-dimensional structural analysis and the reported steel member design utilisation, while retaining the governing station, governing load combination and underlying capacity calculations for review.

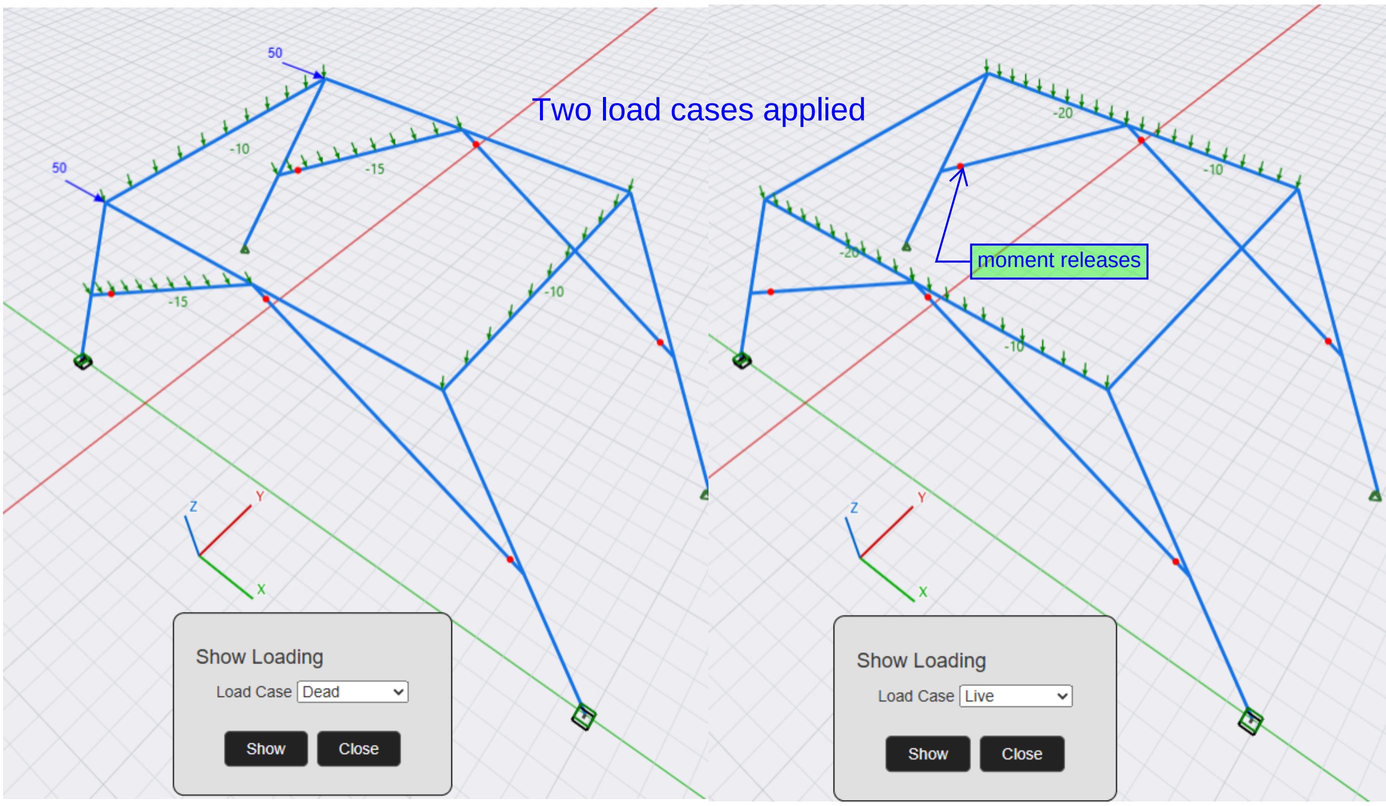
Appendix – Verification Example

Verification example - FEAKalc 3D vs. ETABS

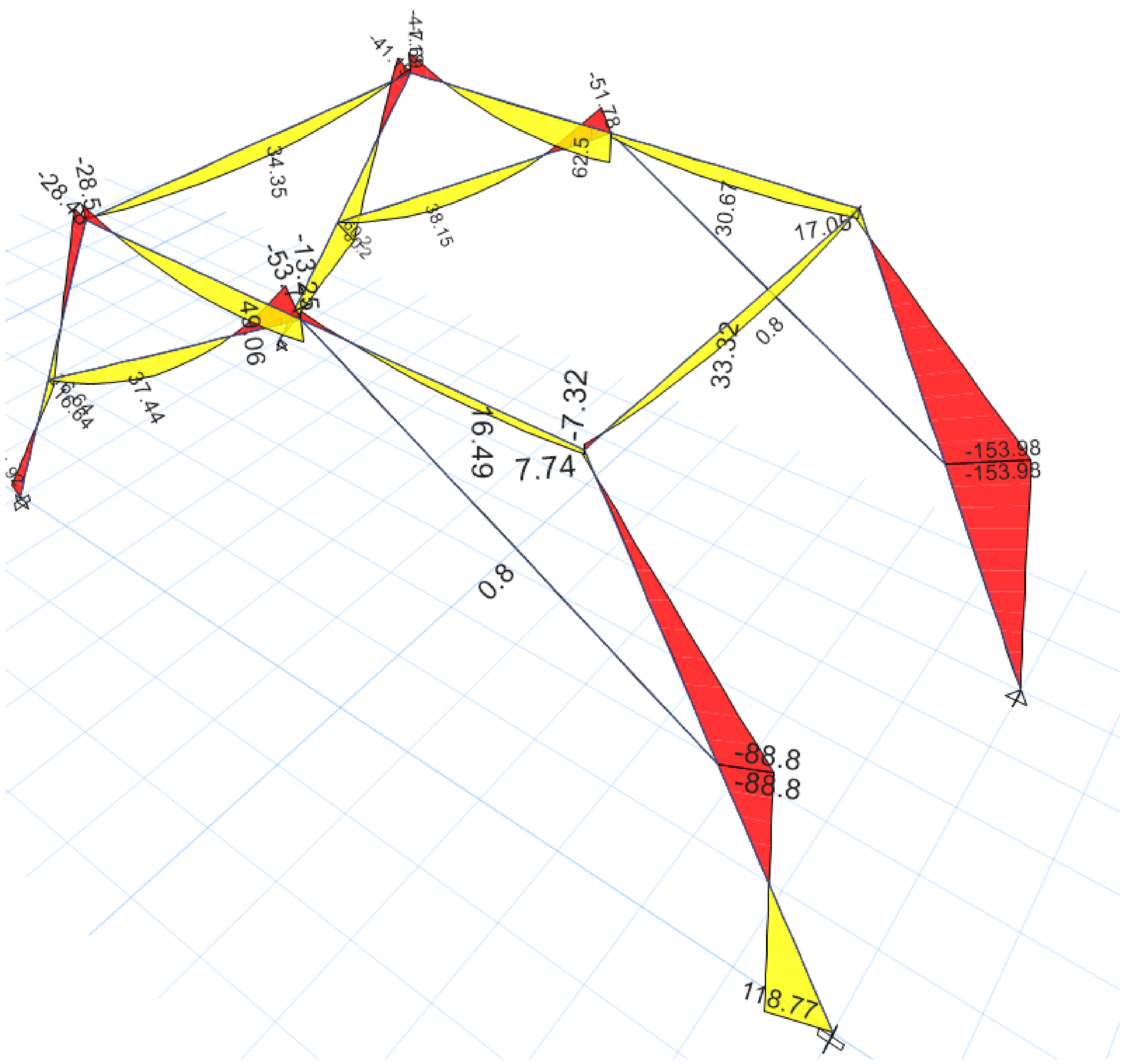
Verification Statement

This example compares FEAKalc 3D directly with ETABS for a representative steel space frame. Independent agreement is demonstrated for structural displacements, internal force distributions and support reactions, providing confidence that FEAKalc accurately reproduces established finite element solutions for three-dimensional frame analysis. The comparison validates the underlying finite element implementation rather than a single numerical result, showing consistent behaviour across the entire structural model





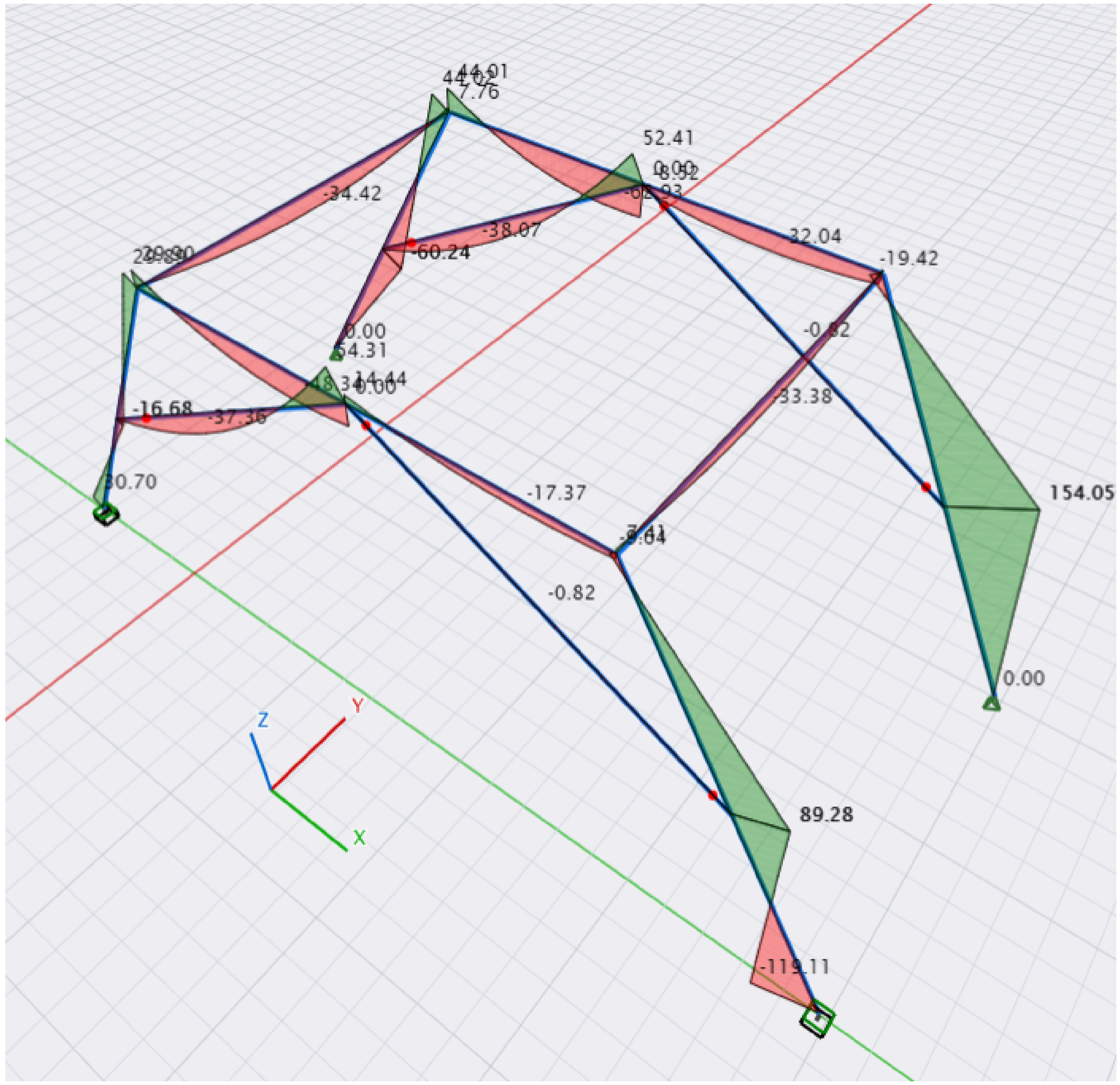
ETABS



Bending Moment Diagram M33

Load Combo : 1.2DL + 1.5LL

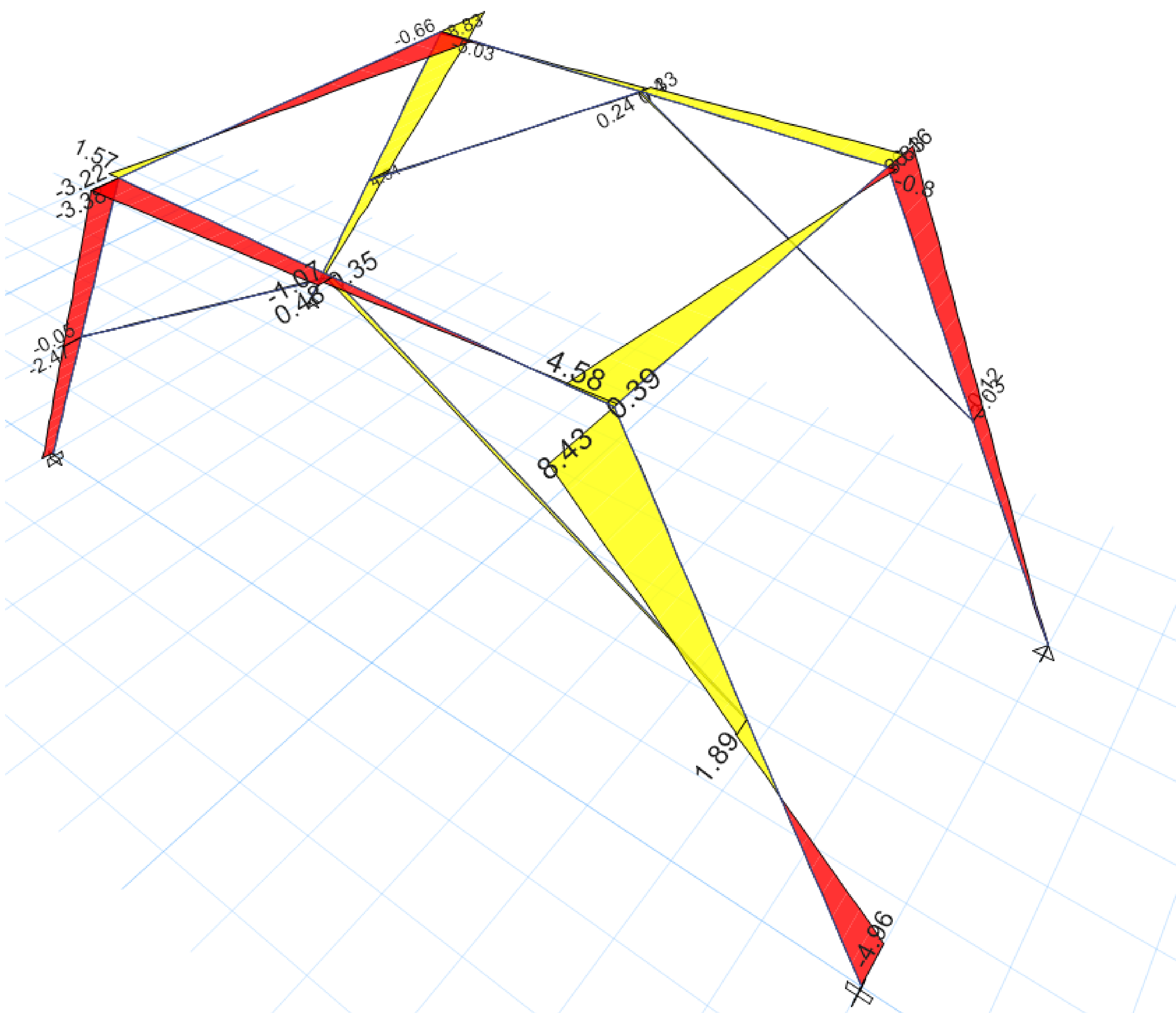
FEAKalc 3D



Bending Moment Diagram M33

Load Combo : 1.2DL + 1.5LL

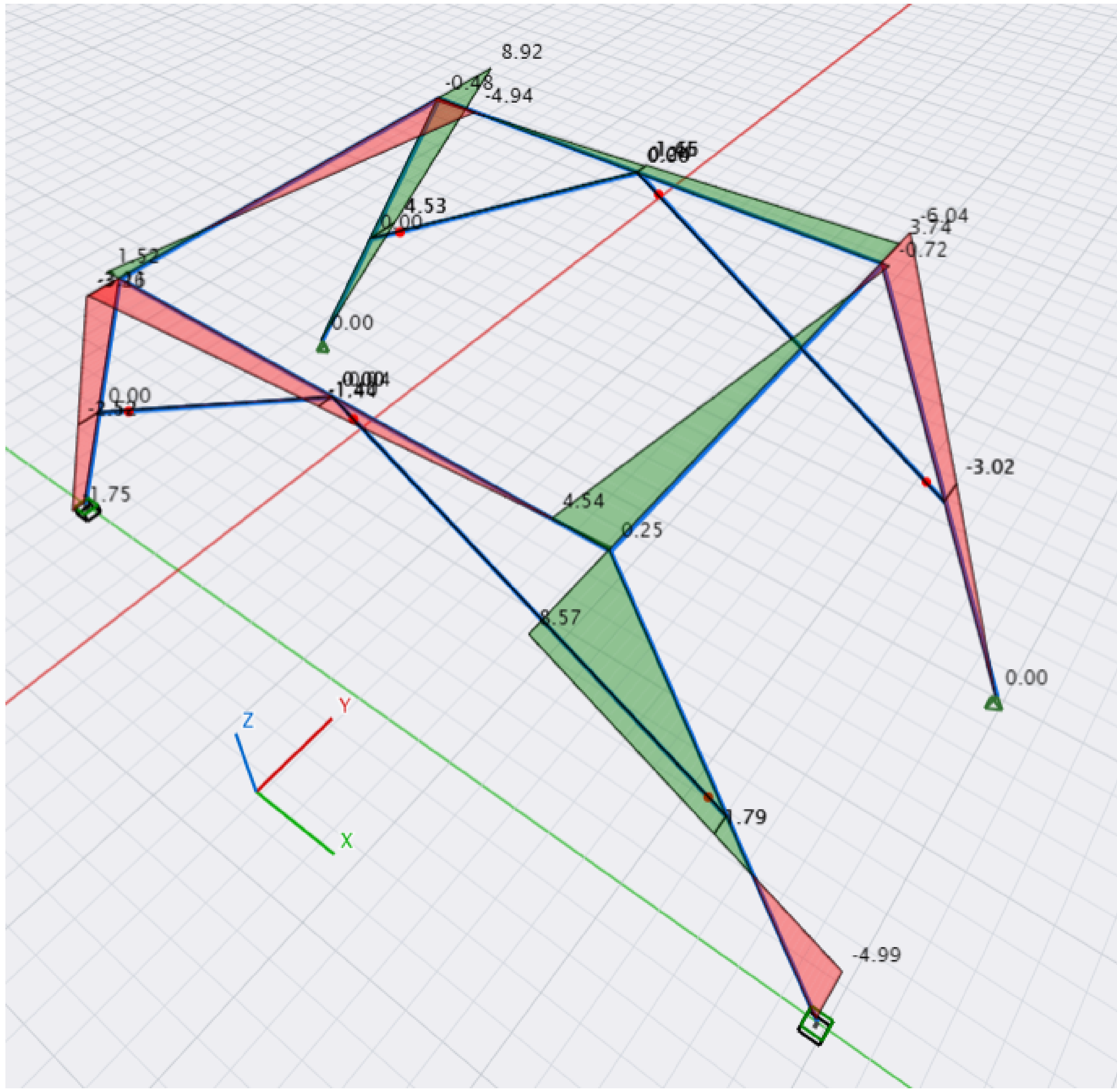
ETABS



Bending Moment Diagram M22

Load Combo : 1.2DL + 1.5LL

FEAKalc 3D

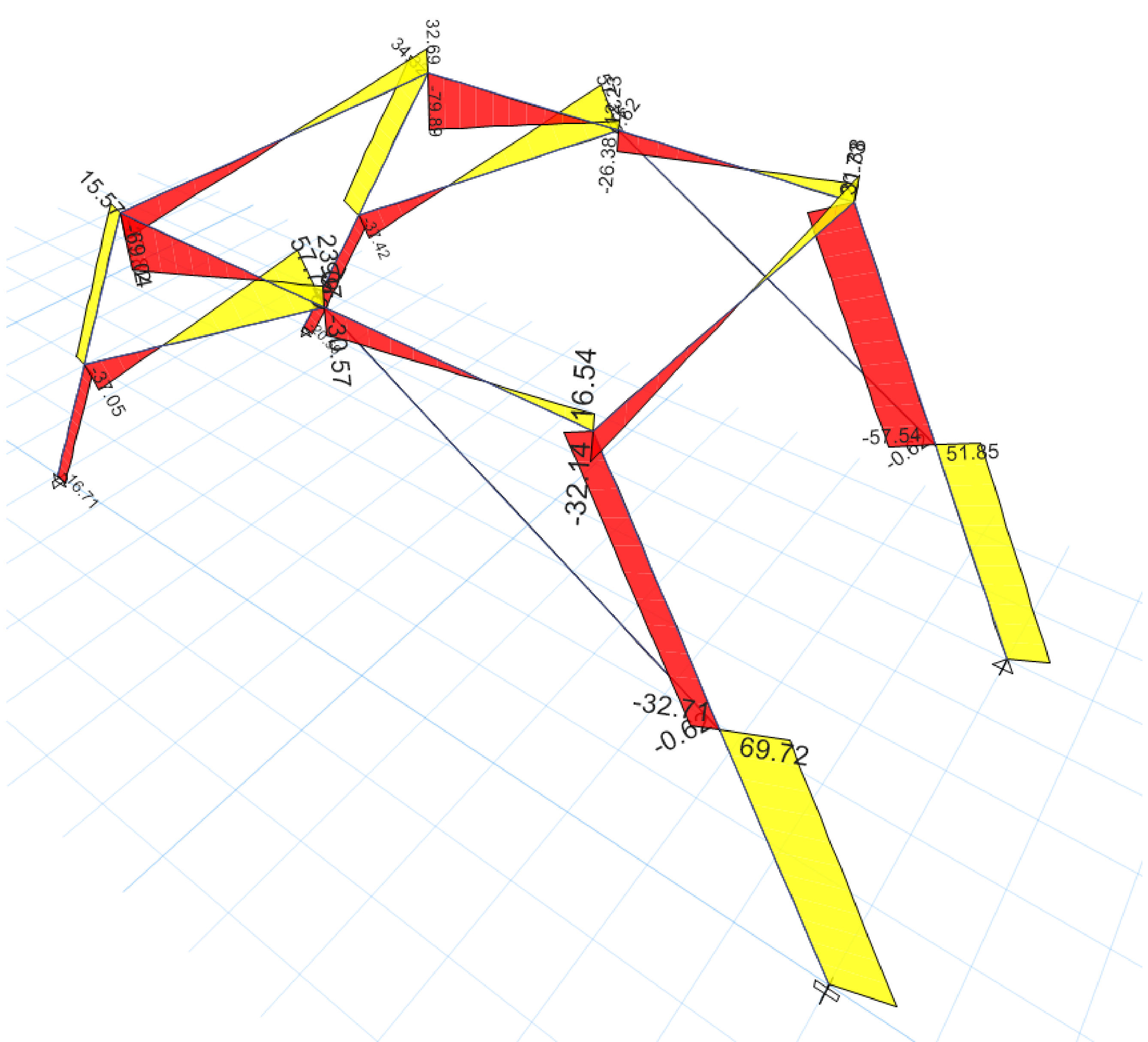


Bending Moment Diagram M22

Load Combo : 1.2DL + 1.5LL

Analysis Verification

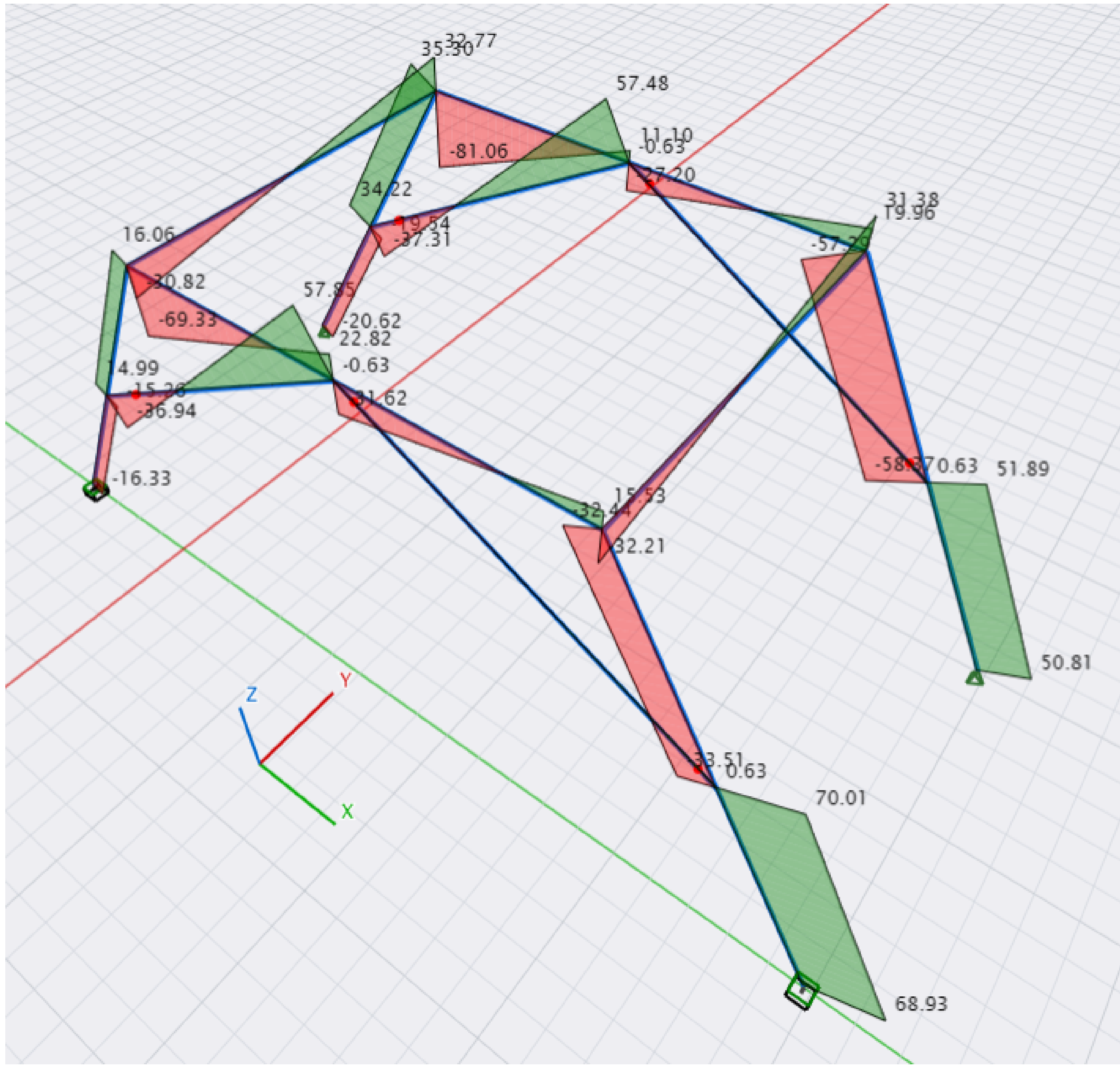
ETABS



Shear Diagram V22

Load Combo : 1.2DL + 1.5LL

FEAKalc 3D

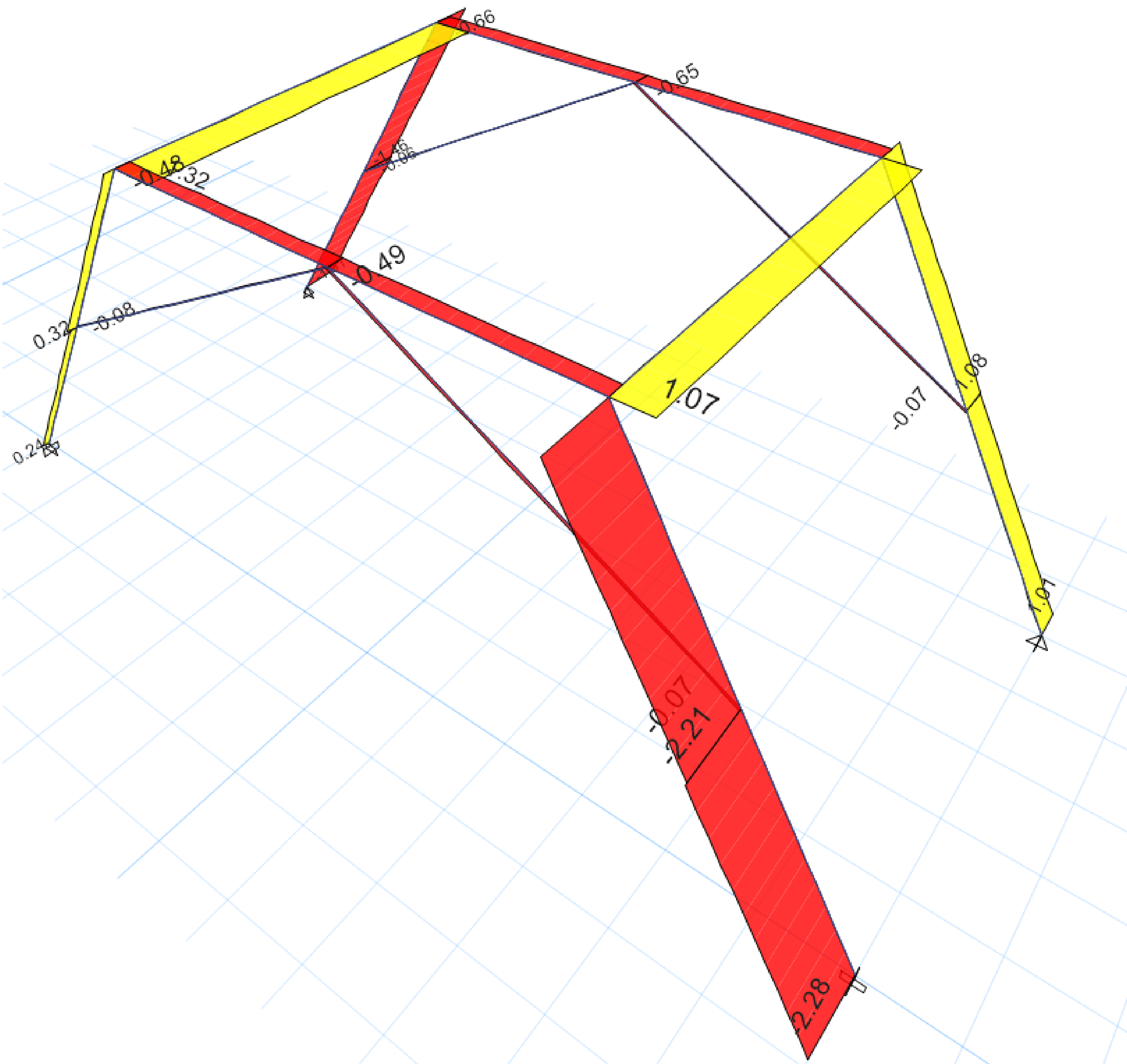


Shear Diagram V22

Load Combo : 1.2DL + 1.5LL

LEARNING CENTRE	Analysis Verification	Concrete Frame	By	RK
		FEAKalc 3D Verifications	Sheet No. 6	Date 7/26

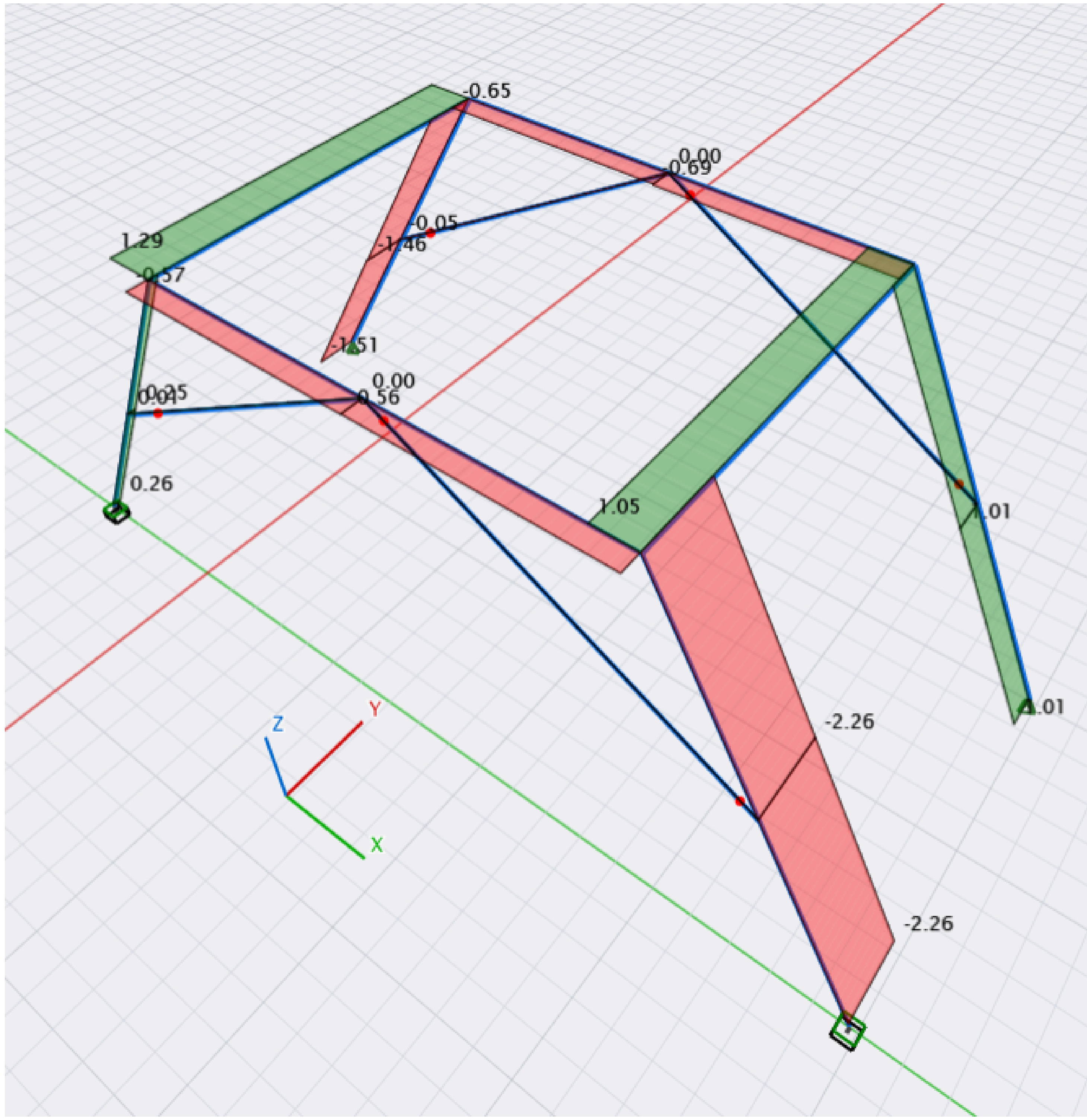
ETABS



Shear Diagram V33

Load Combo : 1.2DL + 1.5LL

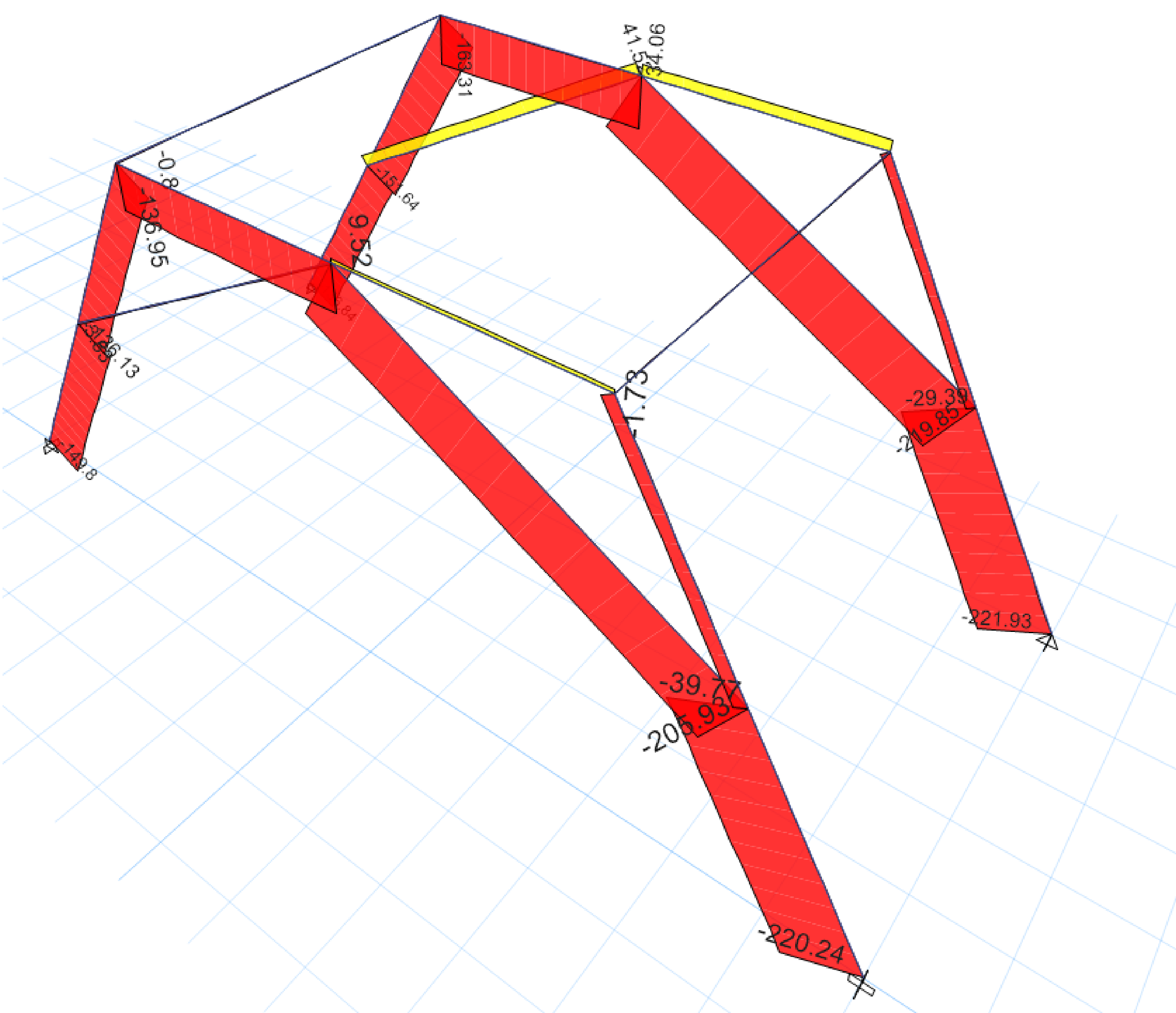
FEAKalc 3D



Shear Diagram V33

Load Combo : 1.2DL + 1.5LL

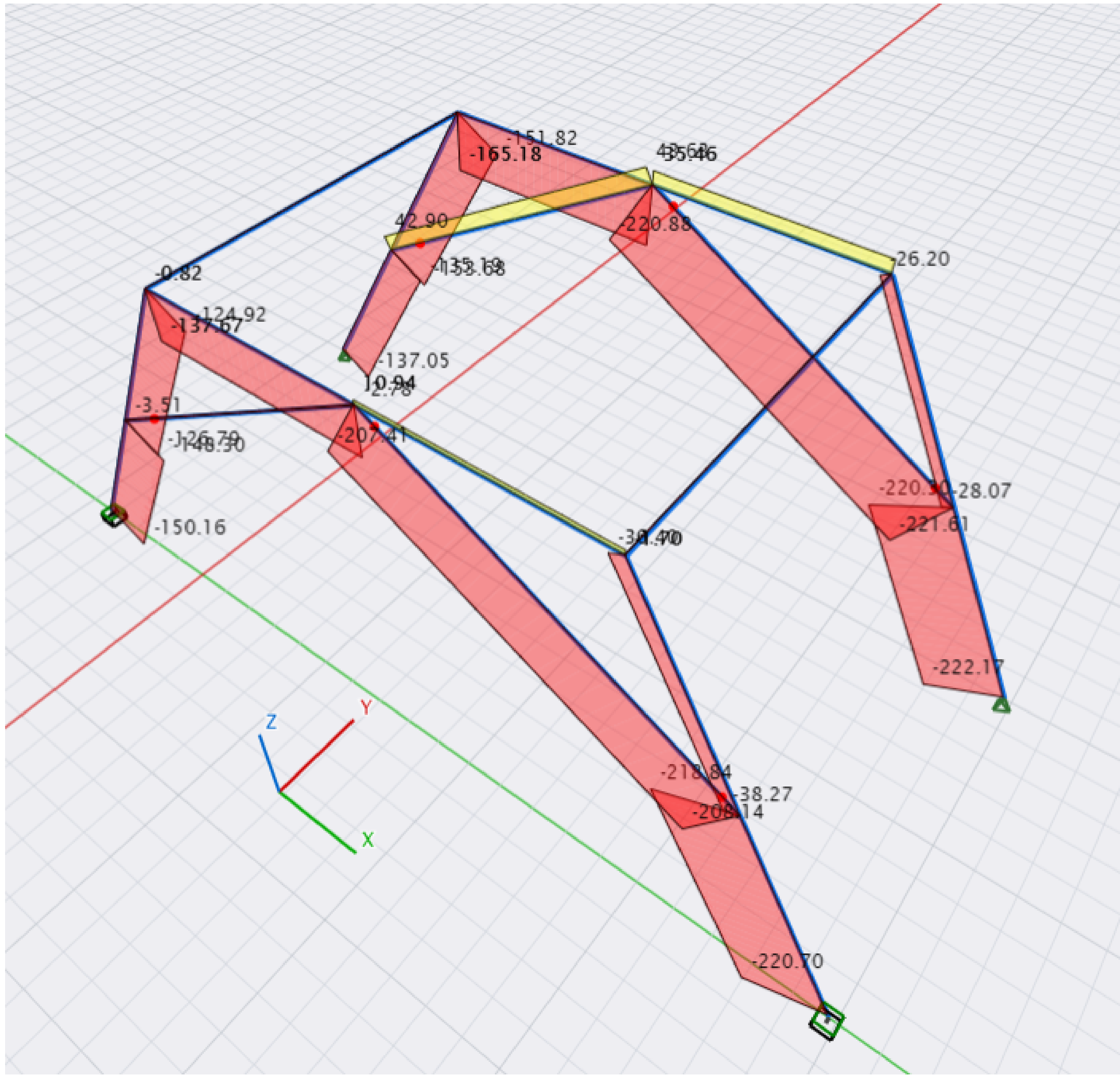
ETABS



Axial Force Diagram

Load Combo : 1.2DL + 1.5LL

FEAKalc 3D

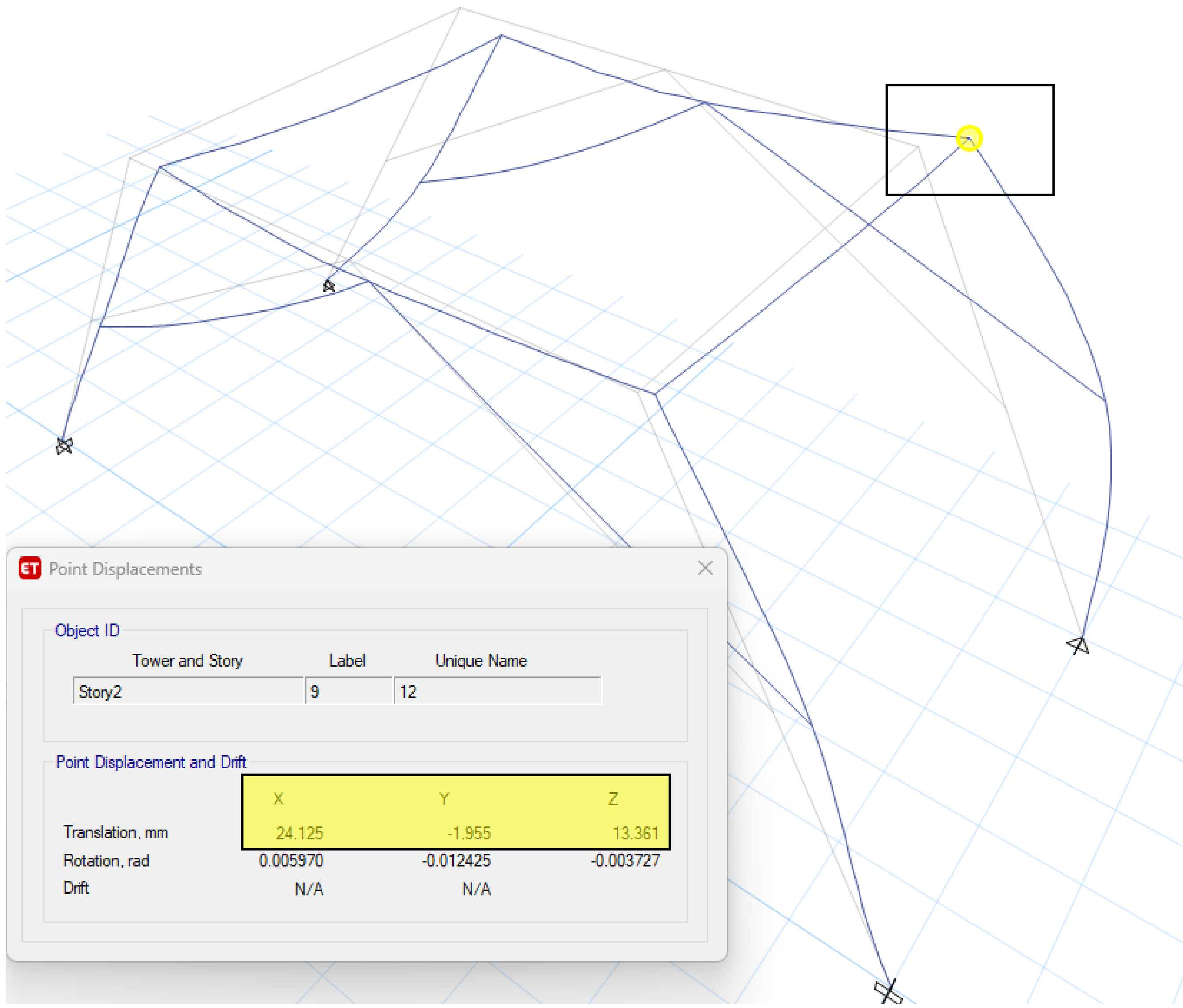


Axial Force Diagram

Load Combo : 1.2DL + 1.5LL

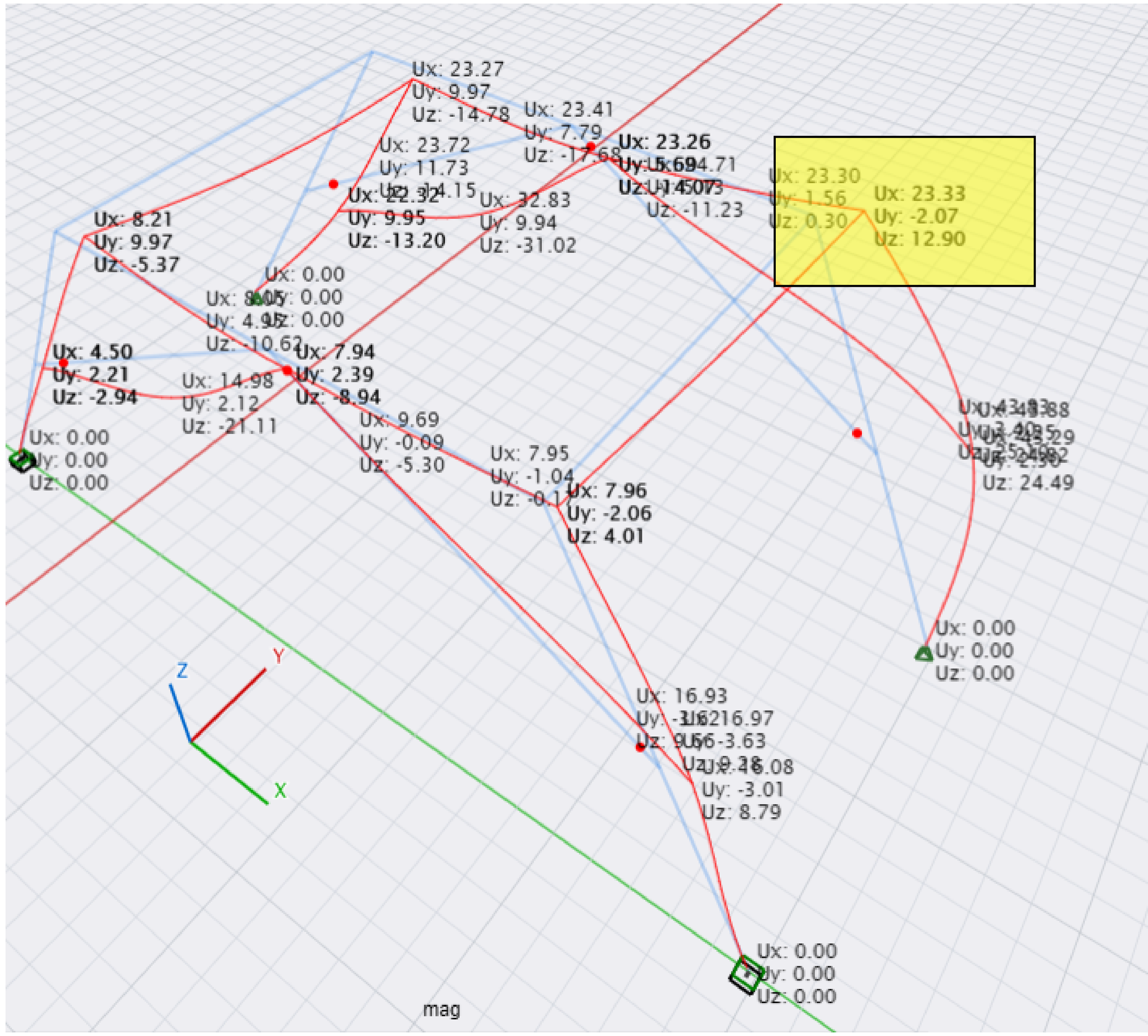
LEARNING CENTRE	Analysis Verification	Concrete Frame	By	RK
		FEAKalc 3D Verifications	Sheet No. 8	Date 7/26

ETABS



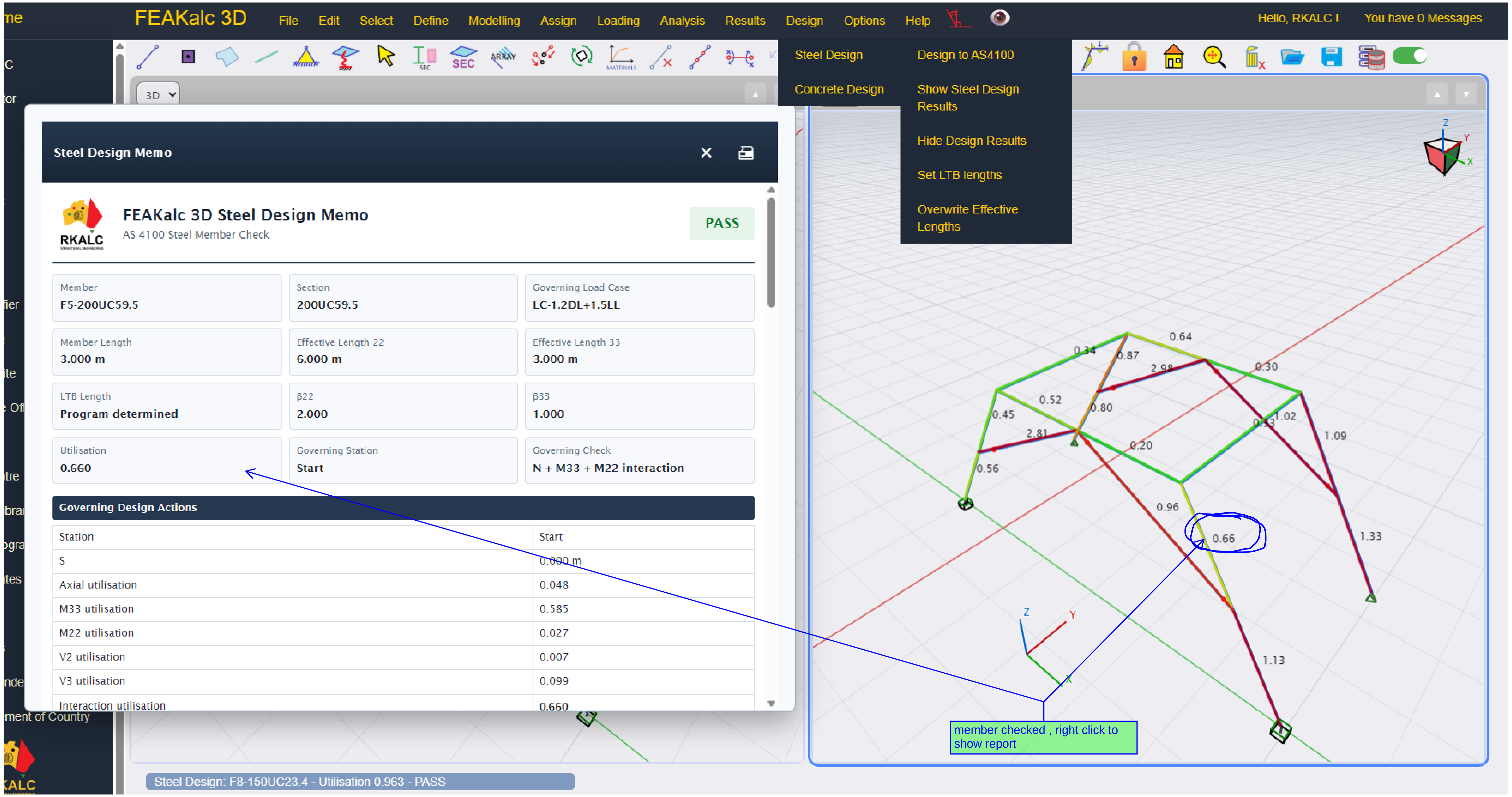
Load Combo : 1.2DL + 1.5LL

FEAKalc 3D



Load Combo : 1.2DL + 1.5LL

Following is user interface - design ratios shown below and deliberately left in case of over-stress, for example



Member being checked is subject to N=38.27 / M22= 1.793/ M33=89.283 / V = 33.515

Member capacities shown below

Global Ratios

At M33 = 38.27 / 153 = 58.3%

At M22 = 1.793/67 = 2.7%

At N = 38.27 / 793 = 4.8%

$\rightarrow$ 66 $\frac{3}{8}$ %

I Sections

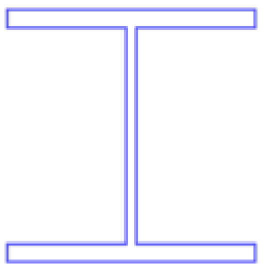
Your Design Package is valid until 30/03/2030 10:44:38 PM

Geometry

length at strong axis

length at weak axis

Steel Grade	300
Section Family	UC
Section	200UC59.5
Length - Axis X (mm)	3000
Length - Axis Y (mm)	6000



Key Properties

fy (N/mm²)	300	fu (N/mm²)	440
Class (X)	Compact	Class (Y)	Compact
Ze (X)	656000	Ze (Y)	298537

Section Properties

Depth	210	Width	205
tw	9.3	tf	14.2
Weight (kg/m)	59.82	Area	7620
Ix	6.13E+07	Iy	2.04E+07
Zx	583809.5	Zy	199024.4
Sx	656000	Sy	303000
rx	89.6918	ry	51.74133
Iw	1.96E+11	J	477000

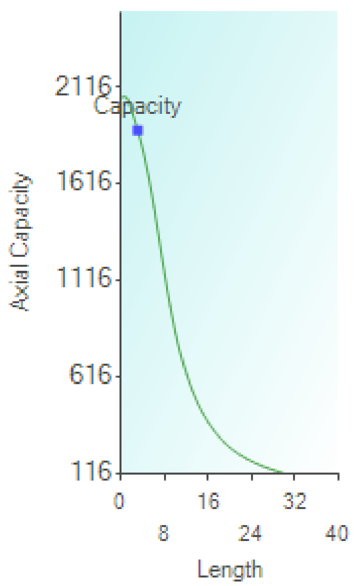
Axis X

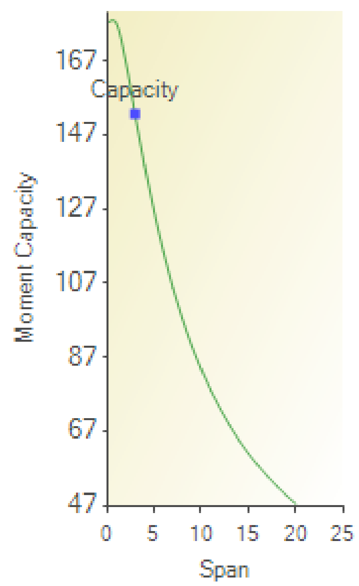
Key Values

αs	0.862	αm	1	Moa	603
λ	36.64	ξ	3.744	αa	17.16
αc	0.918	Kf	1	η	0.075
λn	36.64	αb	0	Mb/Ms	86 %

Capacities

ΦMsx (KN.m)	177	ΦNsx (KN)	2057
ΦMbx (KN.m)	153	ΦNcx (KN)	1889
ΦVweb (KN)	337		





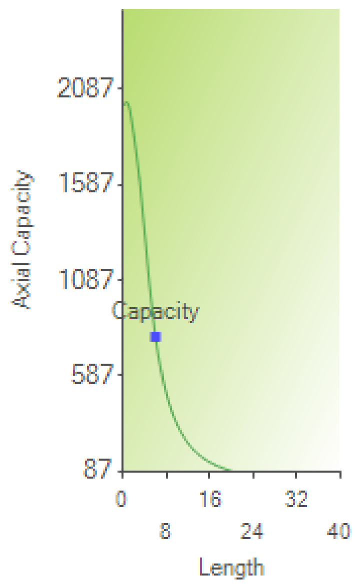
Axis Y

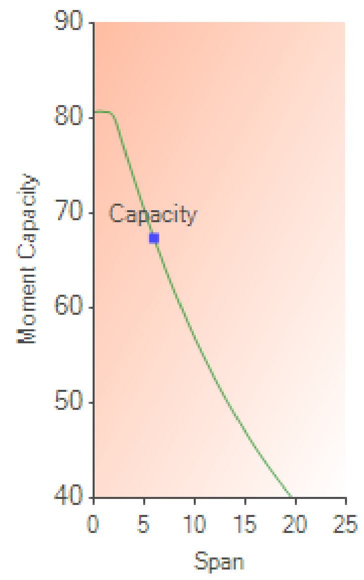
Key Values

αs	0.835	αm	1	Moa	234
λ	127.029	ξ	0.844	αa	14.678
αc	0.385	Kf	1	η	0.37
λn	127.029	αb	0	Mb/Ms	83 %

Capacities

ΦMsy (KN.m)	81	ΦNsy (KN)	2057
ΦMby (KN.m)	67	ΦNcy (KN)	793





FEAKalc 3D Steel Design Memo

AS 4100 Steel Member Check

PASS

Member **F5-200UC59.5**	Section **200UC59.5**	Governing Load Case **LC-1.2DL+1.5LL**
Member Length **3.000 m**	Effective Length 22 **6.000 m**	Effective Length 33 **3.000 m**
LTB Length **Program determined**	β22 **2.000**	β33 **1.000**
Utilisation **0.660**	Governing Station **Start**	Governing Check **N + M33 + M22 interaction**

Governing Design Actions

Station	Start
S	0.000 m
Axial utilisation	0.048
M33 utilisation	0.585
M22 utilisation	0.027
V2 utilisation	0.007
V3 utilisation	0.099
Interaction utilisation	**0.660**

Capacities

Ncx	1889.297 kN
Ncy	793.045 kN
Nc	793.045 kN
Mbx	152.586 kNm
Mby	67.267 kNm
Vv	337.478 kN

Section Classification

Major axis class	Compact
Minor axis class	Compact
λ web	22.092
λ flange	7.549
Kf	1.000

Applied Actions at Stations

Station	S m	N* kN	M33* kNm	M22* kNm	V3* kN	V2* kN	T* kNm
Start	0.000	-38.270	89.283	1.793	-33.515	-2.260	-0.001
Station 2	0.125	-38.193	85.097	2.075	-33.470	-2.260	-0.001
Station 3	0.250	-38.115	80.916	2.358	-33.425	-2.260	-0.001
Station 4	0.375	-38.037	76.741	2.641	-33.380	-2.260	-0.001
Station 5	0.500	-37.959	72.571	2.923	-33.335	-2.260	-0.001
Station 6	0.625	-37.882	68.407	3.206	-33.291	-2.260	-0.001
Station 7	0.750	-37.804	64.249	3.488	-33.246	-2.260	-0.001
Station 8	0.875	-37.726	60.096	3.771	-33.201	-2.260	-0.001
Station 9	1.000	-37.649	55.949	4.053	-33.156	-2.260	-0.001
Station 10	1.125	-37.571	51.807	4.336	-33.111	-2.260	-0.001
Station 11	1.250	-37.493	47.671	4.618	-33.066	-2.260	-0.001
Station 12	1.375	-37.415	43.541	4.901	-33.021	-2.260	-0.001
Middle	1.500	-37.337	39.416	5.184	-32.976	-2.260	-0.001
Station 14	1.625	-37.260	35.297	5.466	-32.931	-2.260	-0.001
Station 15	1.750	-37.182	31.183	5.749	-32.886	-2.260	-0.001
Station 16	1.875	-37.104	27.075	6.031	-32.842	-2.260	-0.001
Station 17	2.000	-37.027	22.973	6.314	-32.797	-2.260	-0.001
Station 18	2.125	-36.949	18.876	6.596	-32.752	-2.260	-0.001
Station 19	2.250	-36.871	14.785	6.879	-32.707	-2.260	-0.001
Station 20	2.375	-36.793	10.700	7.161	-32.662	-2.260	-0.001
Station 21	2.500	-36.716	6.620	7.444	-32.617	-2.260	-0.001
Station 22	2.625	-36.638	2.546	7.726	-32.572	-2.260	-0.001
Station 23	2.750	-36.560	-1.523	8.009	-32.527	-2.260	-0.001
Station 24	2.875	-36.482	-5.586	8.292	-32.483	-2.260	-0.001
End	3.000	-36.405	-9.643	8.574	-32.438	-2.260	-0.001

Station-by-Station Check

Station	S	Status	Util.	Axial	M33	M22	V2	V3	Governing Check
Start	0.000	PASS	**0.660**	0.048	0.585	0.027	0.007	0.099	N + M33 + M22 interaction
Station 2	0.125	PASS	**0.637**	0.048	0.558	0.031	0.007	0.099	N + M33 + M22 interaction
Station 3	0.250	PASS	**0.613**	0.048	0.530	0.035	0.007	0.099	N + M33 + M22 interaction
Station 4	0.375	PASS	**0.590**	0.048	0.503	0.039	0.007	0.099	N + M33 + M22 interaction
Station 5	0.500	PASS	**0.567**	0.048	0.476	0.043	0.007	0.099	N + M33 + M22 interaction
Station 6	0.625	PASS	**0.544**	0.048	0.448	0.048	0.007	0.099	N + M33 + M22 interaction
Station 7	0.750	PASS	**0.521**	0.048	0.421	0.052	0.007	0.099	N + M33 + M22 interaction
Station 8	0.875	PASS	**0.497**	0.048	0.394	0.056	0.007	0.098	N + M33 + M22 interaction
Station 9	1.000	PASS	**0.474**	0.047	0.367	0.060	0.007	0.098	N + M33 + M22 interaction
Station 10	1.125	PASS	**0.451**	0.047	0.340	0.064	0.007	0.098	N + M33 + M22 interaction
Station 11	1.250	PASS	**0.428**	0.047	0.312	0.069	0.007	0.098	N + M33 + M22 interaction
Station 12	1.375	PASS	**0.405**	0.047	0.285	0.073	0.007	0.098	N + M33 + M22 interaction
Middle	1.500	PASS	**0.382**	0.047	0.258	0.077	0.007	0.098	N + M33 + M22 interaction
Station 14	1.625	PASS	**0.360**	0.047	0.231	0.081	0.007	0.098	N + M33 + M22 interaction
Station 15	1.750	PASS	**0.337**	0.047	0.204	0.085	0.007	0.097	N + M33 + M22 interaction
Station 16	1.875	PASS	**0.314**	0.047	0.177	0.090	0.007	0.097	N + M33 + M22 interaction
Station 17	2.000	PASS	**0.291**	0.047	0.151	0.094	0.007	0.097	N + M33 + M22 interaction
Station 18	2.125	PASS	**0.268**	0.047	0.124	0.098	0.007	0.097	N + M33 + M22 interaction
Station 19	2.250	PASS	**0.246**	0.046	0.097	0.102	0.007	0.097	N + M33 + M22 interaction
Station 20	2.375	PASS	**0.223**	0.046	0.070	0.106	0.007	0.097	N + M33 + M22 interaction
Station 21	2.500	PASS	**0.200**	0.046	0.043	0.111	0.007	0.097	N + M33 + M22 interaction
Station 22	2.625	PASS	**0.178**	0.046	0.017	0.115	0.007	0.097	N + M33 + M22 interaction
Station 23	2.750	PASS	**0.175**	0.046	0.010	0.119	0.007	0.096	N + M33 + M22 interaction
Station 24	2.875	PASS	**0.206**	0.046	0.037	0.123	0.007	0.096	N + M33 + M22 interaction
End	3.000	PASS	**0.237**	0.046	0.063	0.127	0.007	0.096	N + M33 + M22 interaction

Design Notes

Compression capacity check based on member effective length.
Bending capacity check based on section effective modulus and member bending capacity.
Interaction check currently reported as axial + M33 + M22 utilisation.
Governing result is taken as the maximum utilisation from all FEAKalc stations.

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