# Lateral torsional buckling Check for facade elements by using FEM analysis

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*Abstract* - **In the façade industry, thin-walled aluminum or steel frame elements are used often with slender open cross-sections with low torsional stiffness. It is generally subjected to a complex loading condition where axial, bending, shear, and torsional stresses may act simultaneously. Due to these thin and slender sections, members may fail in buckling even before the yield strength is reached. This effect refers to lateral torsional buckling (LTB), which requires good understanding of structural behavior which requires highly experienced skills and often ignored by practicing façade engineers. Ignoring this effect or checking may lead to either uneconomical design or in some instances unsafe designs. Hence the engineers need to be familiar with the LTB checking, this paper has tried to illustrate a Software based checking and code validation methods. following are overview of this study. • To study the lateral torsional buckling behavior • The importance of LTB check in various façade systems • Work flow as per EN codal analysis to predict the buckling capacity (**

*keywords* - **Facade engineering, Lateral torsional buckling, Elastic critical bending moment (Mcr), FEM Analysis**

## I. INTRODUCTION

A Lateral torsional buckling (LTB) is a type of structural failure that can occur in a building facade, particularly in long, slender columns or beams. It occurs when the structure is subjected to lateral forces, such as wind or earthquake loads, and the structural members twist or buckle under the load. In facade engineering, LTB is a common concern because it can lead to the collapse of the facade and pose a safety hazard. To prevent LTB, engineers design the structural members with sufficient lateral stability and use bracing or other techniques to resist the lateral forces. Factors that can influence the likelihood of LTB include the material properties of the structural members, the size and shape of the members, and the spacing and orientation of the members. Engineers use computer modelling and analysis techniques to predict the likelihood of this effect and design the structure accordingly.

Preventing LTB is an important part of facade engineering and requires careful consideration of the structural design and load conditions. Glass, aluminium, mild steel, and stainless steel are the most common materials used in façade structures. In the process of structural design, engineers need to consider the effects of instability. It is common to note that many façade elements are slender, and it's structural failures and collapses are due to structural instability, which makes it more difficult and complex to predict the effects.

***Laterally supported beam:***

A beam is a flexure member, and when it is adequately supported against lateral buckling, the beam failure occurs by yielding of the material at the point of maximum moment. The beam is thus capable of reaching its plastic moment capacity under the applied loads. Thus, the design strength is governed by yield stress, and the beam is classified as a laterally supported beam. In this case, LTB is not critical, the designer can optimise the design based on normal bending stress and deflection criteria.

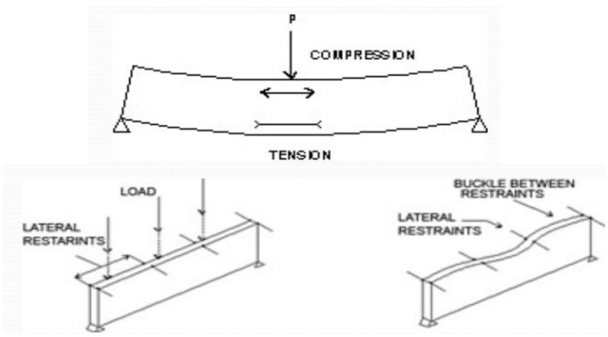


Figure 1: Behaviour of beam with lateral restraint

***Laterally un supported beam:***

A beam is a flexure member, and when it is inadequate to support the lateral deflection and twisting, the beam failures occur by lateral torsional buckling prior to reach of their full plastic moment capacity. As a result, the member load resistance will be greatly reduced, and the collapse may occur suddenly before or after material yielding. When a beam fails by lateral torsional buckling, it buckles about its weak axis, even though it is loaded in a strong plane Such beams are classified as a laterally unsupported beam. In this case, LTB is very critical, and the designer should consider the effects while designing.

Usually, LTB may occur in an unrestrained beam when its compression flange is free to laterally rotate and laterally displace. The lateral-torsion buckling is characterized by the mode of rigid body movements of the whole member in which individual cross-sections rotate and translate but do not distort in shape. To prevent such structural failure, the buckling phenomenon has to be studied.

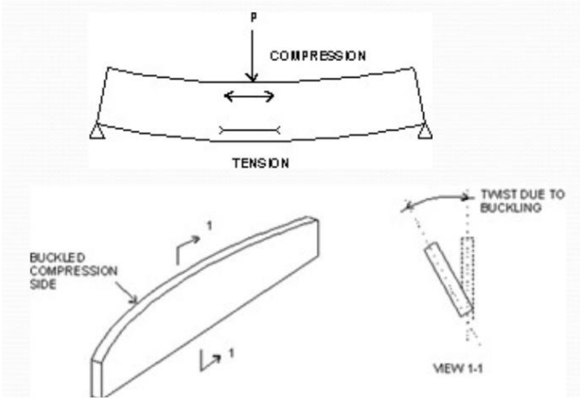


Figure 2: Behaviour of beam without lateral restraint

***Code Limitation***

It is typical for façade profiles to be non-symmetrical, and determining the buckling capacity can be difficult due to limitations in some international approaches, including the EN code-based technique. Calculating the elastic critical moment $M_{cr}$ is a crucial step in the LTB process. However, only double and mono-symmetric prismatic sections are covered by the major international or EN code's calculation process for the $M_{cr}$ value. This article explores the Finite Element Method (FEM) technique utilising software since there are no codal procedures to arrive at $M_{cr}$ for non-symmetrical prismatic sections. The $M_{cr}$ value for various kinds of sections has been calculated for this investigation using the STRAND 7 software

## II. Impact of LTB Check in various façade systems

Dead load, wind load, live load, and temperature load are the major loads while designing the façade system. To achieve the architectural intent (visual), design performance, and comfortable site execution, façade systems are forcefully designed with high slenderness with open cross sections. Due to this, following typical façade systems are critical in LTB.

- Unitized curtainwall system
- Triple height stick curtainwall glazing
- Entrance Fin glazing
- Cantilever Canopy without lateral restraint

***Unitized systems***

Glass has been supported by an aluminium grid work framing system, vertical frames (called "mullion") span between floors, supported by brackets at each floor. Horizontal framing (called "Transom") connected to verticals is used to glaze the glass units. The glazed panels are fully finished at the factory and delivered to the site and ready to be erected in place. Normally, mullion profiles are slender with open cross sections, which is critical for LTB.

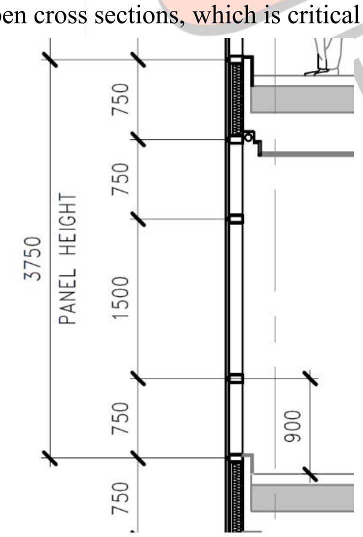


Figure 3: Typical Sections, Actual UCW panels at fabrication

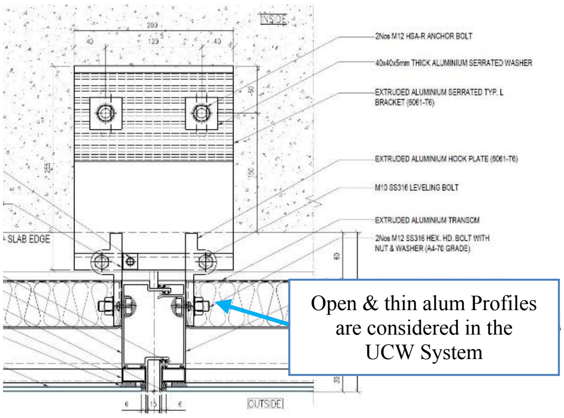


Figure 4: Glass canopy supported with steel structure

***Triple Height stick curtainwall Glazing***

Glass has been supported by vertical aluminium with a steel insert profile (called "mullion") span between floors supported by brackets at the top and bottom. Mullions are manufactured in a factory and delivered to the job site. Mullion is connected to RCC with HDG steel brackets. Glass is delivered frameless and glazed on site. Glass shall be fixed in place using a pressure plate and cover caps using the necessary gaskets, screws, and SS dead load supports. In this system, no horizontal tie members are used. Based on this condition, the mullion slenderness ratio will be higher, which is critical for LTB.

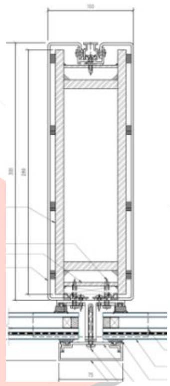

Figure 5: Triple height stick curtain wall system-Actual site installation view

***Frameless fin glazing***

In this entrance frameless glazing, Face Glass is supported by a vertical glass fin plate spanning between floors. The glass fin plate is connected to the RCC with HDG steel brackets at the top and bottom. Normally, the glass fin plate is thin and slender, which is critical for LTB.

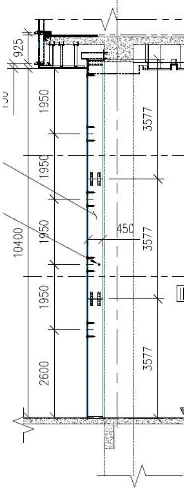


Figure 6: Frameless fin glazing

***Cantilever canopy without lateral restraint***

In this glass canopy, the glass has been supported by a steel structure. The cantilever steel structure is connected to the RCC with HDG steel brackets at the bottom only. Tapered steel profiles are used without lateral support. Hence, LTB check is required for this special structure.

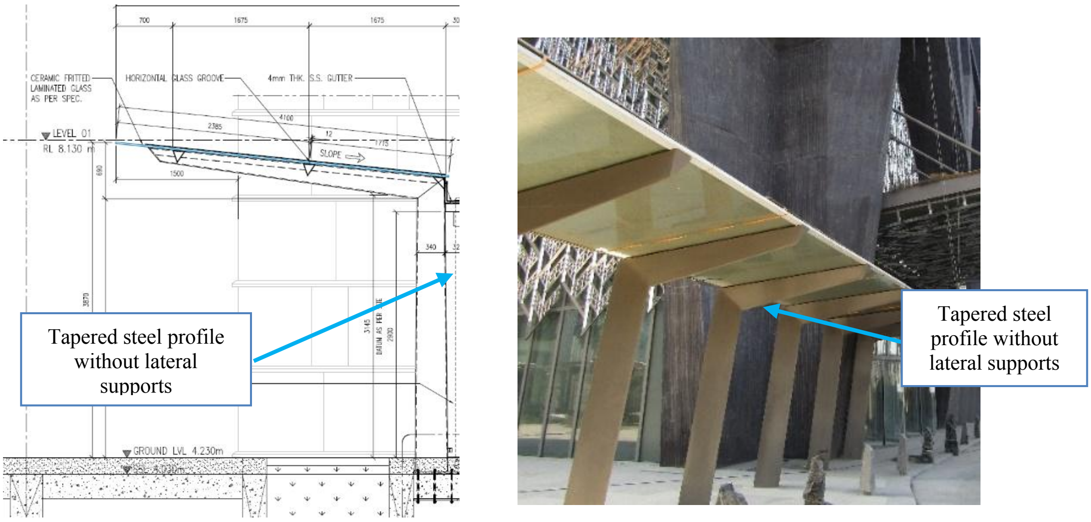


Figure 7: Glass canopy supported with steel structure-Actual site installation view

## III. Lateral Torsional buckling check as stipulated in ENV 1999-1:2007

The following step-by-step procedure to find the buckling capacity of the elements using Code
Step 1: Find the elastic critical bending moment ($M_{cr}$), refer to section 3.3.
Step 2: Find the relative slenderness parameter $\bar{\lambda}_{LT}$, refer to section 3.2.
Step 3: Find the reduction factor for lateral torsional buckling $\chi_{LT}$, Refer section 3.2.
Step 4: Find the the Buckling resistance $M_{b,Rd}$, Refer section 3.1.
Note: For detailed information, refer to EN 1999-1-1:2007- Design of Aluminum Structures.

***Buckling resistance***

Laterally unrestrained member subject to major axis bending shall be verified against lateral-torsional buckling as follows:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1{,}0$$

where:
$M_{Ed}$ is the design value of the bending moment
$M_{b,Rd}$ is the design buckling resistance moment.

The design buckling resistance moment of laterally un-restrained member should be taken as:

$$M_{b,Rd} = \chi_{LT} \alpha w_{el,y} f_0 / \gamma_{M1}$$

where:
$w_{el,y}$ is the elastic section modulus of the gross section, without reduction for HAZ softening, local buckling or holes.
$\alpha$ is taken from Table 6.4 subject to the limitation $\alpha \leq w_{pl,y}/w_{el,y}$.
$\chi LT$ is the reduction factor for lateral torsional buckling.

***Reduction for lateral torsional buckling***

The reduction factor for lateral torsional buckling $\chi$LT for the appropriate relative slenderness $\bar{\lambda}_{LT}$ should be determined from

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{LT}^2}} \quad \text{but } \chi_{LT} \leq 1$$

Where:
$\phi_{LT} = 0{,}5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{0,LT}) + \bar{\lambda}_{LT}^2]$
$\alpha_{LT}$ is an imperfection factor
$\bar{\lambda}_{LT}$ is the relative slenderness
$\bar{\lambda}_{0,LT}$ is the limit of the horizontal plateau
$M_{cr}$ is the elastic critical moment for lateral-torsional buckling.

The relative slenderness parameter $\bar{\lambda}_{LT}$ should be determined from

$$\bar{\lambda}_{LT} = \sqrt{\frac{\alpha w_{el,y} f_{\theta}}{M_{cr}}}$$

***Elastic Critical moment-$M_{cr}$***

*Uniform cross section with both symmetrical axis*

The elastic critical moment for lateral-torsional buckling of a beam of uniform symmetrical cross-section with equal flanges, under standard conditions of restraint at each end and subject to uniform moment in plane going through the shear center is given by ANNEX-I

$$M_{cr} = \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{L^2 G I_t}{\pi^2 E I_z} + \frac{I_w}{I_z}}$$

where:

$$G = \frac{E}{2(1+\nu)}$$

$I_t$ is the torsional constant

$I_w$ is the warping constant

$I_z$ is the second moment of area about the minor axis

$L$ is the length of the beam between points that have lateral restraint

$\nu$ is the Poisson ratio

*Uniform cross section with major/minor symmetry*

In the case of a beam of uniform cross-section which is symmetrical about the minor axis, for bending about the major axis the elastic critical moment for lateral-torsional buckling is given by the general formula:

$$M_{cr} = \mu_{cr} \frac{\pi \sqrt{E I_Z G I_t}}{L}$$

Where relative non-dimensional critical moment $\mu_{cr}$ is

$$\mu_{cr} = \frac{C_1}{k_Z}\left[\sqrt{1 + K_{wt}^2 + (C_2\zeta_g - C_3\zeta_j)^2} - (C_2\zeta_g - C_3\zeta_j)\right]$$

Non-dimensional torsion parameter is $K_{wt} = \frac{\pi}{k_w L}\sqrt{\frac{E I_w}{G I_t}}$

relative non-dimensional coordinate of the point of load application related to shear $\zeta_g = \frac{\pi z_g}{k_z L}\sqrt{\frac{E I_z}{G I_t}}$

relative non-dimensional cross-section mono-symmetry parameter $\zeta_j = \frac{\pi z_j}{k_z L}\sqrt{\frac{E I_z}{G I_t}}$

where:

$C_1$, $C_2$ and $C_3$ are factors depending mainly on the loading and end restraint conditions

$k_Z$ and $k_w$ are buckling length factors

$$z_g = z_a - z_s$$

$$z_j = z_s - \frac{0,5}{I_y}\int_A (y^2 + z^2) z \, dA$$

$z_a$ is the coordinate of the point of load application related to centroid

$z_s$ is the coordinate of the shear related to centroid

$z_g$ is the coordinate of the point of load application related to shear.

## IV. Lateral Torsional buckling check as per finite element method simulation technique

As stated above, code-based approach focused for few types of profiles. For complex and un-symmetrical profiles, following step-by-step procedure illustrated to find buckling capacity using FEM Technique

Step 1: Find the eigen value & elastic critical bending moment ($M_{cr}$) from strand7 linear buckling analysis.

Step 2: Find the relative slenderness parameter $\bar{\lambda}_{LT}$, refer to section 3.2.

Step 3: Find the reduction factor for lateral torsional buckling $\chi_{LT}$, Refer section 3.2.

Step 4: Find the the Buckling resistance $M_{b,Rd}$, Refer section 3.

***Software brief***
For this study, Strand7 software a general frame analysis with advanced tools for the creation of finite element models has been used. This software has all standard features for pre-processing and post processing stages. Post processing tools for the investigation of results includes deformed displays, data export, animation etc, as like any other software. The solver includes basic Linear Static, Load Influence and Linear Buckling Analysis, a range of Dynamic Analysis solvers including direct and mode superposition solvers, advanced Nonlinear Static and Nonlinear Dynamic solvers and both Steady State and Transient Heat solvers.

***Modelling import various file types***
The software allows for the import of various file types. For the import of CAD geometry, IGES, SAT & STEP formats are supported. These contain the geometric information used by auto meshing to automatically generate a finite element model.
The geometry shown below was built in 3D CAD and imported as a IGES File.

Figure 8: Geometry model from strand7

***Eigen Value approach***
The eigenvalue obtained from linear buckling analysis is a measure of the critical buckling load, or the load at which a structure buckles. This eigenvalue is usually expressed in terms of a non-dimensional number, which is the ratio of the critical buckling load to the applied load. The eigenvalue can also be expressed as a ratio of the critical buckling load to the applied load, and the structure's stiffness. The eigenvalue is the natural frequency of the buckled structure.

***Calculate the eigen value by using linear buckling analysis***
The Linear Buckling Solver calculates the buckling load factors and corresponding mode shapes for a structure under given loading conditions. The linear buckling analysis assumes the existence of a bifurcation point where the primary and secondary loading paths intersect (point A in the figure below). At this point, more than one equilibrium position is possible. The primary path is not usually followed after loading exceeds this point and the structure is in the post-buckling state. The slope of the secondary path at the bifurcation point determines the nature of the post-buckling.
A linear buckling solution is obtained by solving the following eigenvalue problem:

$$[K]\ \{x\} = \lambda\ [K_g]\ \{x\}$$

Where,
[K] Global stiffness matrix
{x} Buckling mode vectors
$\lambda$ Buckling load factor
[Kg] Global geometric stiffness matrix

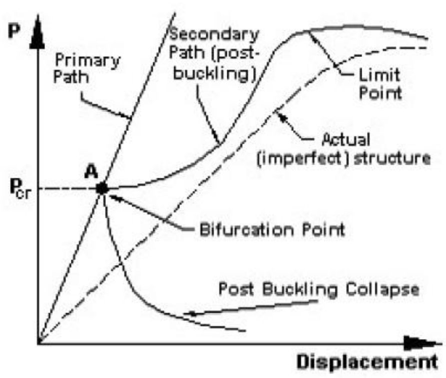


Figure 9: Graph between buckling load vs Displacement

***Calculate the elastic critical moment ($M_{cr}$) by using eigen values***

The eigenvalues are the ratio between the critical buckling load and the applied load as follows:
Eigenvalue/ buckling load factor = buckling load / applied load

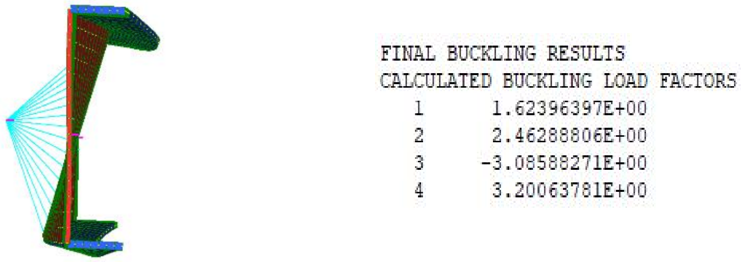


Figure 10: Example image-extracted from Strand7analysis- Buckling modes with eigen values

## V. Case studies and comparison

Various types of profiles were studied and results were compared as below

***5.1 Case study-1- LTB comparison b/w EN code vs FEM for major axis symmetrical section***

A simply supported "C" Channel aluminium member of 2.9m span with a uniformly distributed load of 1.26 kN/m applied at the full length of the span with different depths has been selected to evaluate the buckling capacity. The calculation is done manually using the procedure described in EN CODE. The same beam was then analyzed with Strand7 software. Both Buckling capacity results are tabulated below.

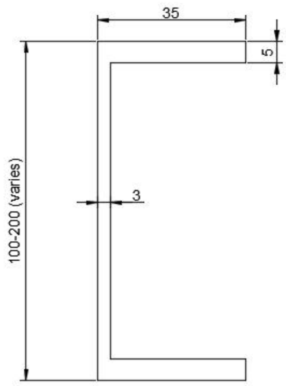


Figure 11: "C" Channel section considered for analysis

Table 1

| $M_{b,Rd}$ For C-Section | | | |
|---|---|---|---|
| ***Section*** | $M_{b,Rd}$ ***Strand (kNm)*** | $M_{b,Rd}$ ***Theoretical (kNm)*** | ***Percentage Error*** |
| C-100 | 0.710 | 0.642 | 9.58 |
| C-125 | 0.770 | 0.728 | 5.5 |
| C-150 | 0.840 | 0.816 | 2.86 |
| C-175 | 0.860 | 0.744 | 13.5 |

$$\text{Percentage Error} = \left|\frac{(M_{b,Rd})\text{Strand} - (M_{b,Rd})\text{Theor}}{(M_{cr})\text{Strand}}\right| \times 100$$

***5.2. Case study-2- LTB comparison b/w EN code vs FEM for non-symmetrical mullion section***

A simply supported " aluminium mullion" member of 2.9m span with a uniformly distributed load of 1.26 kN/m applied at the full length of the span with different depths has been selected to evaluate the buckling capacity. The calculation is done manually using the procedure described in EN CODE. The same beam was then analyzed with Strand7 software. Both Buckling capacity results are tabulated below.

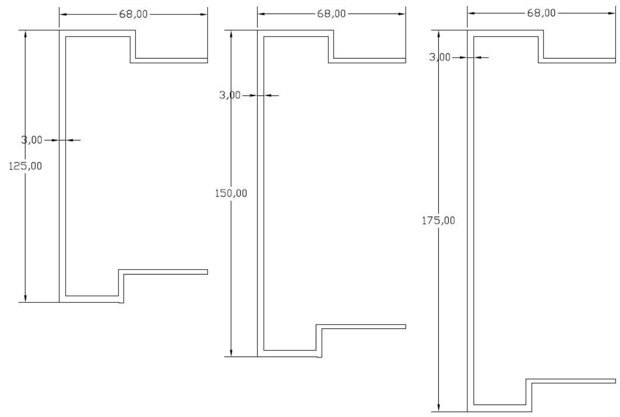


Figure 12: "Aluminium mullion" section considered for analysis

Table 2

| $M_{b,Rd}$ For C-Section | | | |
|---|---|---|---|
| ***Section*** | ***$M_{b,Rd}$ Strand (kNm)*** | ***$M_{b,Rd}$ Theoretical (kNm)*** | ***Percentage Error*** |
| Mullion-125 | 1.198 | 1.240 | 3.39 |
| Mullion-150 | 1.480 | 1.550 | 4.52 |
| Mullion-175 | 1.495 | 1.650 | 9.39 |

$$\text{Percentage Error} = \left|\frac{(M_{b,Rd})\text{Strand} - (M_{b,Rd})\text{Theor}}{(M_{cr})\text{Strand}}\right| \times 100$$

### *5.3. Case study-3- LTB check for Non symmetrical Mullion profile by using FEM technique.*

For the analysis, simply supported aluminium mullions (open, semi-open, and closed profile) were used with a 1.2 kN/m distributed load on various span lengths (1.5m, 2m, 2.5m, 3m, 3.5m, 4m). The I-value of all the profiles is 300 $cm^4$. Buckling capacity is found using STRAND7 software. The comparison of mullion buckling capacity is shown in the below table.

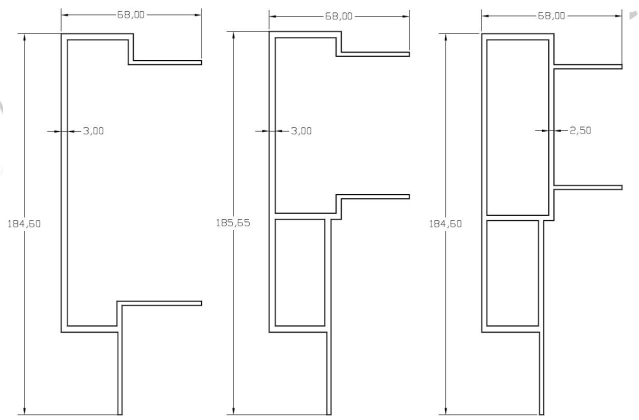


Figure 13: Open, Semi open & closed profiles considered for Analysis

Table 3

| **Length (m)** | **Buckling capacity (kNm)** | | |
|---|---|---|---|
| | ***Open*** | ***Semi-Open*** | ***Closed*** |
| 1.5 | 2.080 | 5.210 | 5.760 |
| 2 | 1.600 | 5.090 | 5.700 |
| 2.5 | 1.310 | 4.960 | 5.630 |
| 3 | 1.100 | 4.820 | 5.550 |
| 3.5 | 0.950 | 4.670 | 5.470 |
| 4 | 0.830 | 4.510 | 5.390 |

## VI. Results and Discussion

***Case study-1***

In the 1st case study as shown above, buckling capacity of various "C" channel aluminium sections has been evaluated as per EN 1999-1-1:2007, and the results have been validated using Strand7software, which employs the FEM technique.

- The difference in results between the two methods was 5.5-13.5%.
- The results obtained from EN CODE stipulation appears to be slightly on conservative basis, has opportunities to further optimize and lead to improve overall structural efficiency.

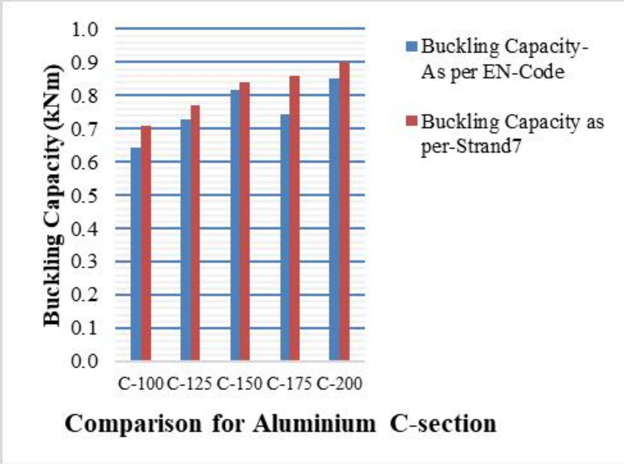


Figure 14: Comparison Graph for buckling capacity between EN Code vs FEM.

***Case study-2***

In the 2nd case study as shown above, buckling capacity of various "Mullion" aluminium sections has been evaluated as per EN 1999-1-1:2007, and the results have been validated using Strand7software, which employs the FEM technique.

- The difference in results between the two methods was 3.4-9.4%.
- The results obtained from EN CODE stipulation are on the safer side for the design purpose but this may lead to economical loss.

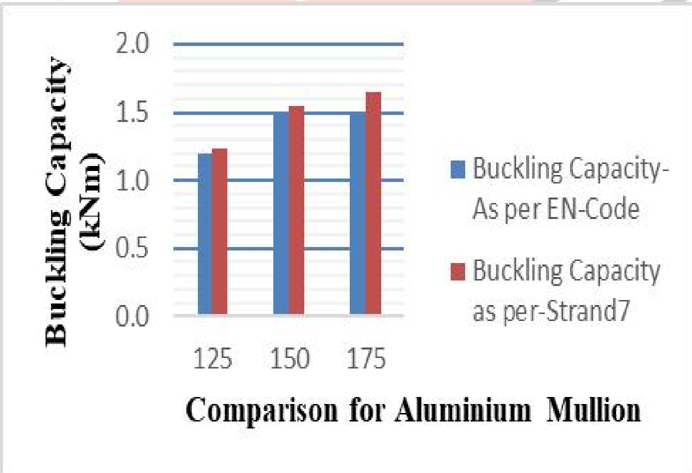


Figure 15: Comparison Graph for buckling capacity between EN Code vs FEM.

***Case study-3***

In the 3rd case study, buckling capacity has been evaluated for three different aluminium mullion sections with different lengths, and it was done using Strand7 software, which works on the FEM technique.

- As the length of the mullion increases with a constant cross section, it results in a reduction in buckling capacity.
- Noted open-profile mullion sections are not feasible for lateral torsional buckling.
- Closed mullion profiles are good for lateral torsional buckling.
- To increase the buckling capacity, the flange thickness should be thicker than the web thickness.

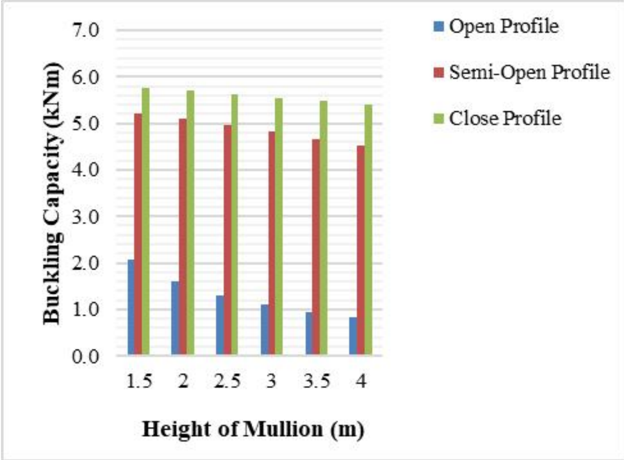


Figure 16: Comparison Graph for buckling capacity between open profile vs Semi-open profile vs Close profile.

## VII. Conclusion

Based on the detailed study, following points are evident.

- It is noted that codes doesn't provide formulation to calculate the $M_{cr}$ for non-symmetrical section & non prismatic section. For challenging and complex profiles, FEM technique found the simplified procedure for the detailed optimization & it is possible to verify the buckling strength of non-symmetrical and non-prismatic profiles with good level of accuracy.
- Using the FEM technique, it is able to take into account both the precise load application point and the geometry profiles that accurately reflect the conditions. Results from buckling capacity are therefore more accurate than those from codal analysis.
- For complex and nonstandard profiles, FEM techniques as demonstrated above is highly optimized and gives reliable approach.
- Following shall be considered as a thumb rule approach for a Unitized Curtain wall profile designs,

⇒ Follow open mullion profile if the unbraced length is less than 1.5 m.
⇒ Follow the semi-open mullion profile if the unbraced length is between 1.5m to 2.4m.
⇒ Follow the closed mullion profile if the unbraced length is more than 2.4 m.

## VIII. Acknowledgment

We thank BES Consultants for supporting this research and also our colleagues who provided their views and opinion that greatly assisted in this research.

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