# STATİK HESAP
# STATIC CALCULATION

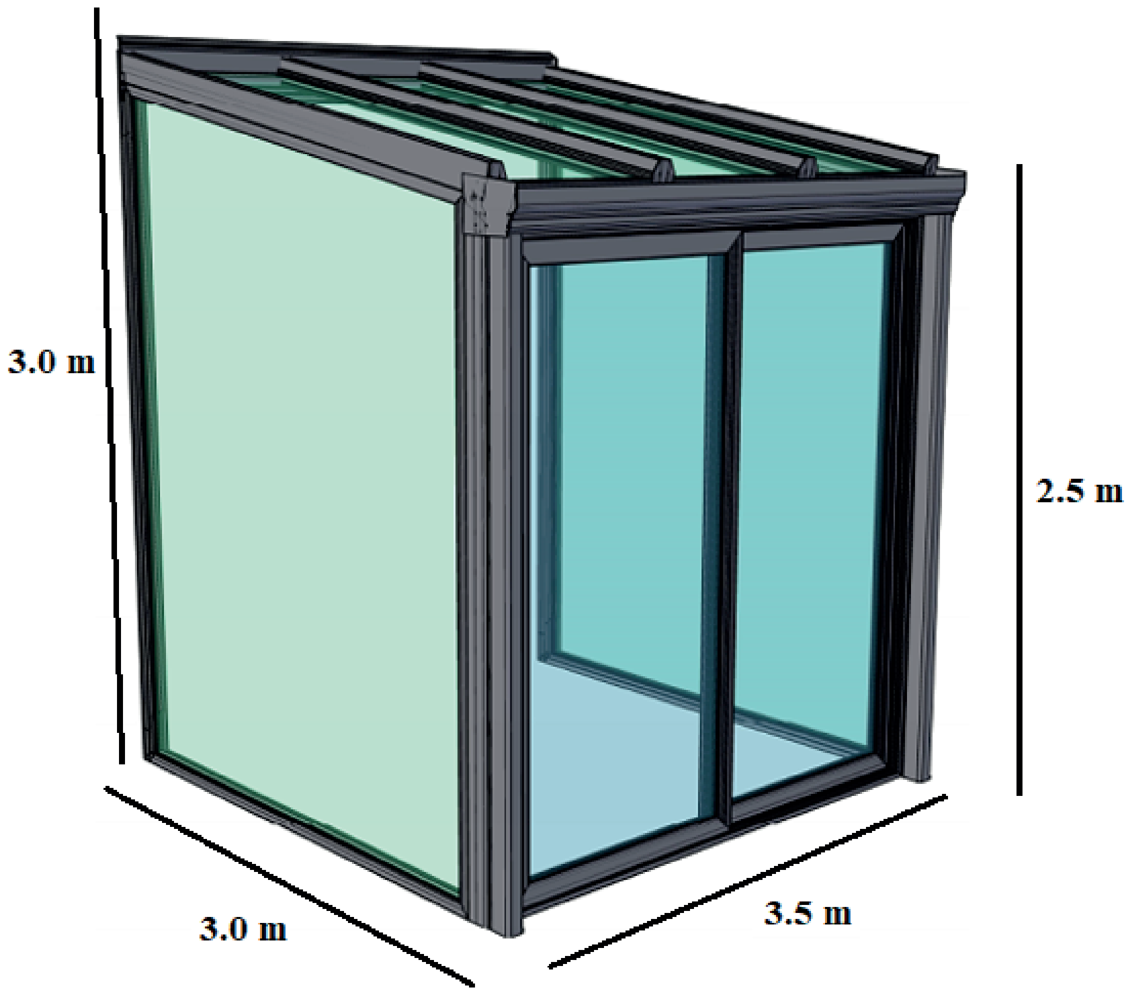
ALUSKY

## 1. SİSTEM GENEL BİLGİLERİ

Aşağıda detayları verilen rapor; Yavuz Metal Alüminyum A.Ş'nin isteği üzerine hazırlanmıştır. Aşağıda çizimleri verilen kış bahçesinin; dış tarafta 6mm temperli, iç tarafta 4 + 4mm çift cam ve $m^2$ de 100kg kar yüküne göre hesapları yapılmıştır.

Statik hesaplamalarda, kış bahçesinin düşey yükler altındaki taşıma kapasitesine bakılmıştır. Rüzgâr ve deprem yüklemeleri yapılmamıştır. Kış bahçesinin uygulanacağı yerin konumuna göre yatay yükler altındaki dayanımına da bakılmalıdır.

Yapının taşıyıcı sistemi SAP2000 V.19.2.1 programı ile analiz edilmiştir. Sistem üç boyutlu olarak modellenmiştir. Hazırlanan bilgisayar modelinde tüm taşıyıcı profiller çubuk elemanlar ile tariflenmiştir.




Şekil 1 Sistem Genel Görünüşü

## 1.1 Yapısal Eleman Malzemeleri

Projede kullanılacak yapısal elemanların malzeme özellikleri aşağıda belirtilmiştir.

### 1.1.1 Yapısal Alüminyum

**Table 3.2b - Characteristic values of 0,2% proof strength $f_o$ and ultimate tensile strength $f_u$ (unwelded and for HAZ), min elongation $A$, reduction factors $\rho_{o,haz}$ and $\rho_{u,haz}$ in HAZ, buckling class and exponent $n_p$ for wrought aluminium alloys - Extruded profiles, extruded tube, extruded rod/bar and drawn tube**

| Alloy EN-AW | Product form | Temper | Thickness $t$ mm 1) 3) | $f_o$ 1) | $f_u$ 1) | $A$ 5) 2) | $f_{o,haz}$ 4) | $f_{u,haz}$ 4) | HAZ-factor 4) | | BC 6) | $n_p$ 7) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | N/mm² | | % | N/mm² | | $\rho_{o,haz}$ | $\rho_{u,haz}$ | | |
| 5083 | ET, EP,ER/B | O/H111, F, H112 | $t \le 200$ | **110** | **270** | 12 | 110 | 270 | 1 | 1 | B | 5 |
| | DT | H12/22/32 | $t \le 10$ | 200 | 280 | 6 | 135 | 270 | 0,68 | 0,96 | B | 14 |
| | | H14/24/34 | $t \le 5$ | 235 | 300 | 4 | | | 0,57 | 0,90 | A | 18 |
| 5454 | ET, EP,ER/B | O/H111 F/H112 | $t \le 25$ | 85 | 200 | 16 | 85 | 200 | 1 | 1 | B | 5 |
| 5754 | ET, EP,ER/B | O/H111 F/H112 | $t \le 25$ | 80 | 180 | 14 | 80 | 180 | 1 | 1 | B | 6 |
| | DT | H14/ H24/H34 | $t \le 10$ | 180 | 240 | 4 | 100 | 180 | 0,56 | 0,75 | B | 16 |
| 6060 | EP,ET,ER/B | T5 | $t \le 5$ | 120 | 160 | 8 | 50 | 80 | 0,42 | 0,50 | B | 17 |
| | EP | | $5 < t \le 25$ | 100 | 140 | 8 | | | 0,50 | 0,57 | B | 14 |
| | ET,EP,ER/B | T6 | $t \le 15$ | **140** | **170** | 8 | 60 | 100 | 0,43 | 0,59 | A | 24 |
| | DT | | $t \le 20$ | 160 | 215 | 12 | | | 0,38 | 0,47 | A | 16 |
| | EP,ET,ER/B | T64 | $t \le 15$ | 120 | 180 | 12 | 60 | 100 | 0,50 | 0,56 | A | 12 |
| | EP,ET,ER/B | T66 | $t \le 3$ | 160 | 215 | 8 | 65 | 110 | 0,41 | 0,51 | A | 16 |
| | EP | | $3 < t \le 25$ | 150 | 195 | 8 | | | 0,43 | 0,56 | A | 18 |
| 6061 | EP,ET,ER/B | T4 | $t < 25$ | 110 | **180** | 15 | 95 | 150 | 0,86 | 0,83 | B | 8 |
| | DT | | $t \le 20$ | 110 | **205** | 16 | | | | 0,73 | B | 8 |
| | EP,ET,ER/B | T6 | $t < 25$ | 240 | **260** | **8** | 115 | 175 | 0,48 | 0,67 | A | 55 |
| | DT | | $t \le 20$ | 240 | **290** | 10 | | | | 0,60 | A | 23 |
| 6063 | EP,ET,ER/B | T5 | $t \le 3$ | 130 | 175 | 8 | 60 | 100 | 0,46 | 0,57 | B | 16 |
| | EP | | $3 < t \le 25$ | 110 | 160 | **7** | | | 0,55 | 0,63 | B | 13 |
| | EP,ET,ER/B | T6 | $t \le 25$ | **160** | **195** | **8** | 65 | 110 | 0,41 | 0,56 | A | 24 |
| | DT | | $t \le 20$ | 190 | 220 | 10 | | | 0,34 | 0,50 | A | 31 |
| | EP,ET,ER/B | T66 | $t \le 10$ | 200 | 245 | **8** | 75 | 130 | 0,38 | 0,53 | A | 22 |
| | EP | | $10 < t \le 25$ | 180 | 225 | 8 | | | 0,42 | 0,58 | A | 21 |
| | DT | | $t \le 20$ | 195 | 230 | 10 | | | 0,38 | 0,57 | A | 28 |
| 6005A | EP/O, ER/B | T6 | $t \le 5$ | 225 | 270 | 8 | 115 | 165 | 0,51 | 0,61 | A | 25 |
| | | | $5 < t \le 10$ | 215 | 260 | 8 | | | 0,53 | 0,63 | A | 24 |
| | | | $10 < t \le 25$ | 200 | 250 | 8 | | | 0,58 | 0,66 | A | 20 |
| | EP/H, ET | T6 | $t \le 5$ | **215** | **255** | 8 | | | 0,53 | 0,65 | A | 26 |
| | | | $5 < t \le 10$ | **200** | **250** | 8 | | | 0,58 | 0,66 | A | 20 |
| 6106 | EP | T6 | $t \le 10$ | 200 | 250 | 8 | 95 | 160 | 0,48 | 0,64 | A | 20 |

## 1.2 Yönetmelik ve Şartnameler

TS 498: Yapı Elemanlarının Boyutlandırılmasında Alınacak Yüklerin Hesap Değerleri

EUROCODE 9: Design of Aluminium Structures

EN 14024: Metal Profiles With Termal Barrier

## 1.3 Yapı Tasarımı Yükleme Kombinasyonları

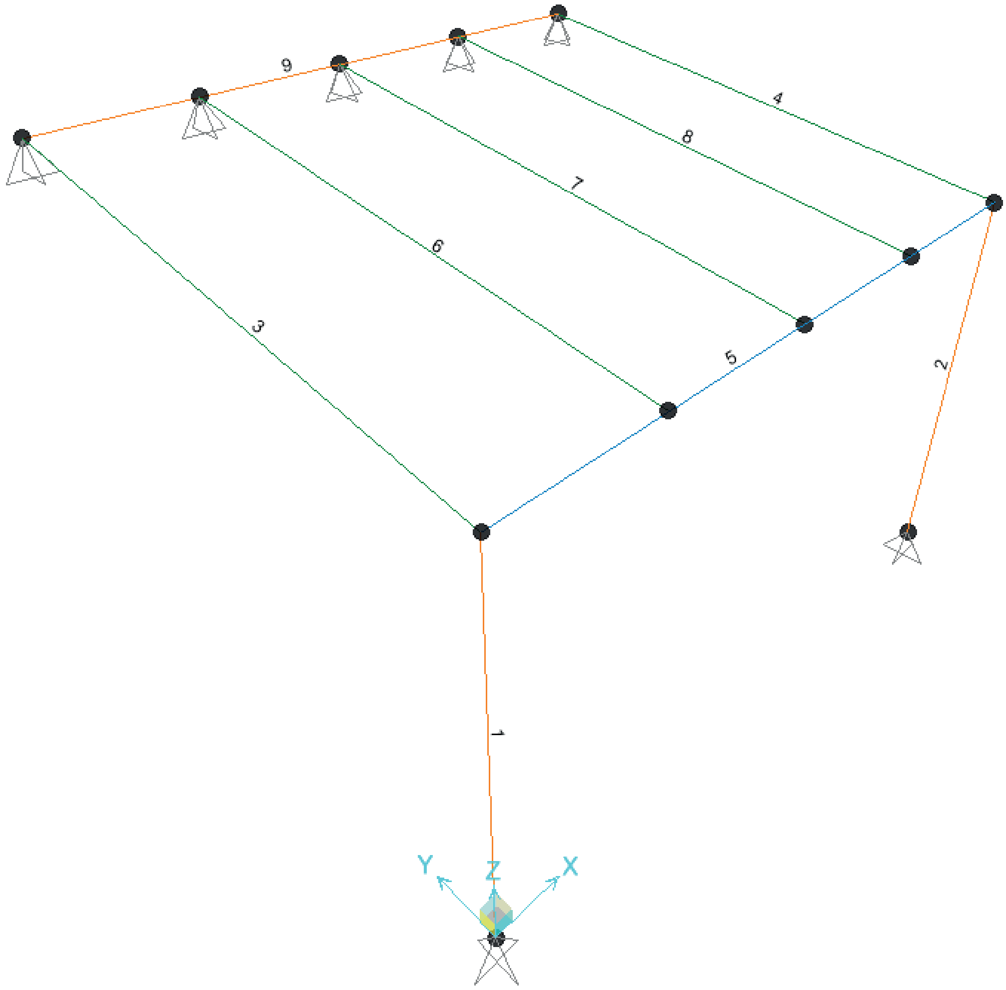
Elemanların boyutlandırılmasında kullanılan yükleme kombinasyonları aşağıda verilmiştir.

G : Sabit Yük

S : Kar Yükü

| TABLE: Combination Definitions | | | | | |
|---|---|---|---|---|---|
| **ComboName** | **ComboType** | **AutoDesign** | **CaseType** | **CaseName** | **ScaleFactor** |
| Text | Text | Yes/No | Text | Text | Unitless |
| EC_01_1.35G | Linear Add | No | Lin. Static | G1 | 1,35 |
| EC_01_1.35G | | | Lin. Static | G2 | 1,35 |
| EC_02_1.35G+1.5S | Linear Add | No | Lin. Static | G1 | 1,35 |
| EC_02_1.35G+1.5S | | | Lin. Static | G2 | 1,35 |
| EC_02_1.35G+1.5S | | | Lin. Static | S | 1,5 |

## 1.4 Sistem Çubuk Eleman Numaraları




Şekil 2 Sistem 3D Modeli

## 2. YÜK ANALİZİ

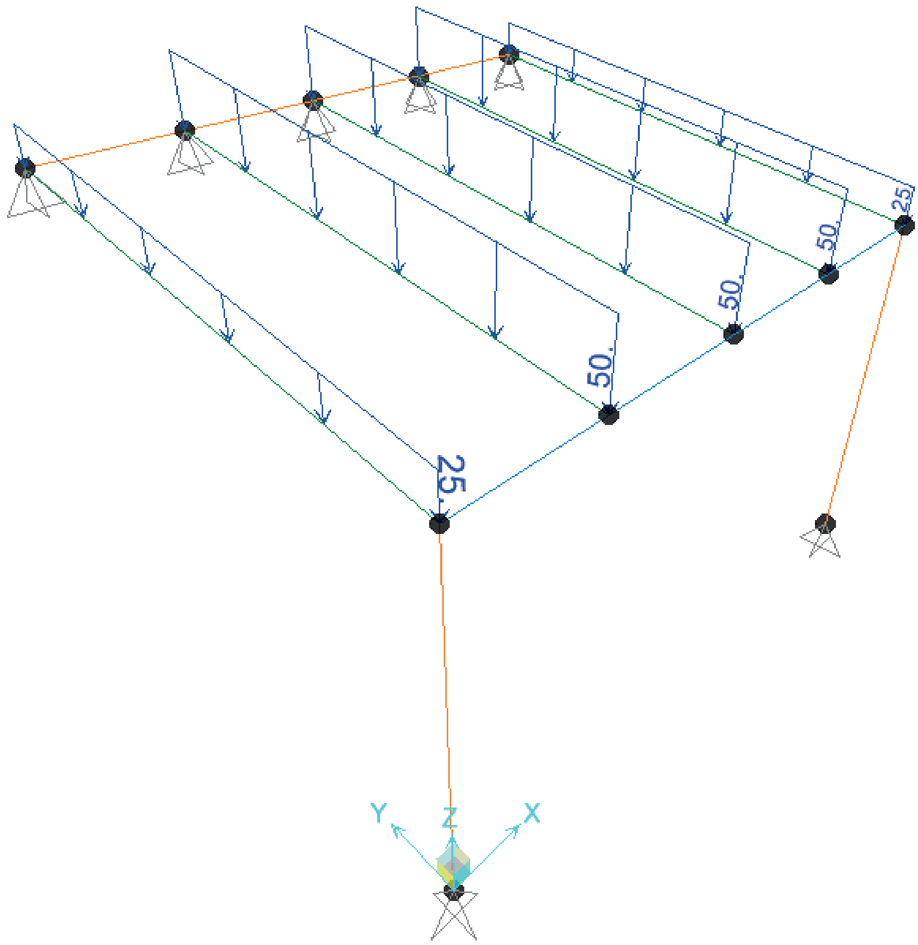
Taşıyıcı sistem profillerinin zati ağırlıkları program tarafından otomatik olarak hesaba katılmış, geri kalan yükler ise giriş bilgisi olarak ilgili düğüm noktalarına ya da çubuk elemanlara uygun şekilde etkitilmiştir.

### 2.1 Sabit Yükler

6 mm temperli dış cam

4 mm + 4 mm lamine cam

Toplam yük: 45 kg/$m^2$




Şekil 3 Sisteme Etkiyen Sabit Yükler

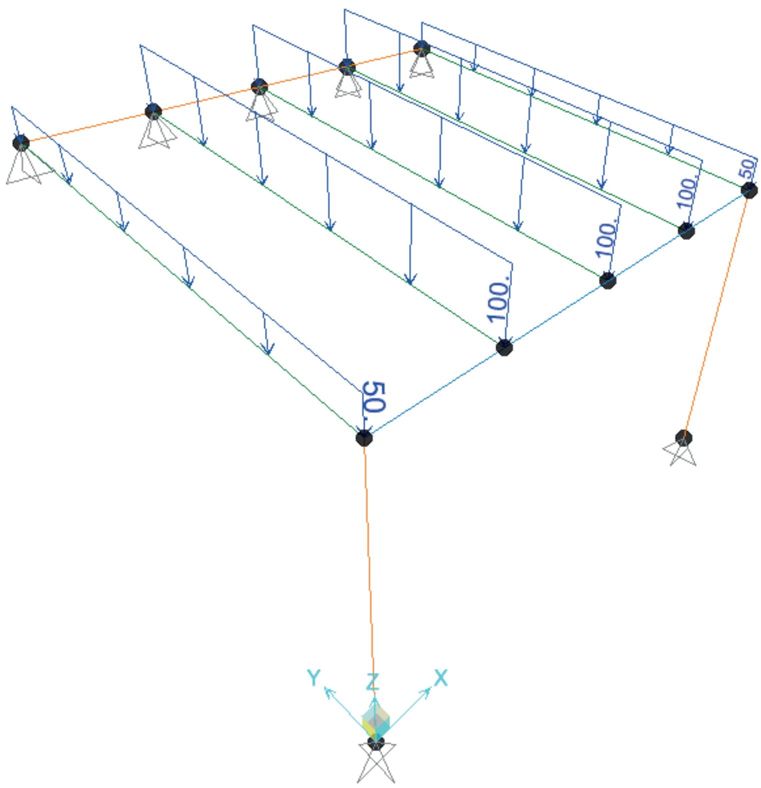
## 2.2 Kar Yükü

| | |
|---|---|
| Rakım | : 700 m |
| Kar bölgesi | : I, II, III, IV |
| Kar yükü | : 100 kg/m$^2$ |

| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| 1 | Yapı yerinin denizden yüksekliği | BÖLGELER | | | |
| | m | I | II | III | IV |
| | ≤ 200 | 0,75 | 0,75 | 0,75 | 0,75 |
| 2 | 300 | 0,75 | 0,75 | 0,75 | 0,80 |
| | 400 | 0,75 | 0,75 | 0,75 | 0,80 |
| | 500 | 0,75 | 0,75 | 0,75 | 0,85 |
| 3 | 600 | 0,75 | 0,75 | 0,80 | 0,90 |
| | 700 | 0,75 | 0,75 | 0,85 | 0,95 |
| | 800 | 0,80 | 0,85 | 1,25 | 1,40 |
| 4 | 900 | 0,80 | 0,95 | 1,30 | 1,50 |
| | 1000 | 0,80 | 1,05 | 1,35 | 1,60 |
| 5 | > 1000 | 1000 m'ye tekabül eden değerler, 1500 m'ye kadar %10, 1500 m'den yukarı yüksekliklerde %15 artırılır. | | | |

Şekil 4 TS 498 Kar Yükü Değerleri

Kar yükü hesap değeri 100 kg/m2 olarak alınmıştır. Bu değer I, II, III ve IV. Bölgelerde maksimum 700 m rakımda olan yerler için kullanılabilir.



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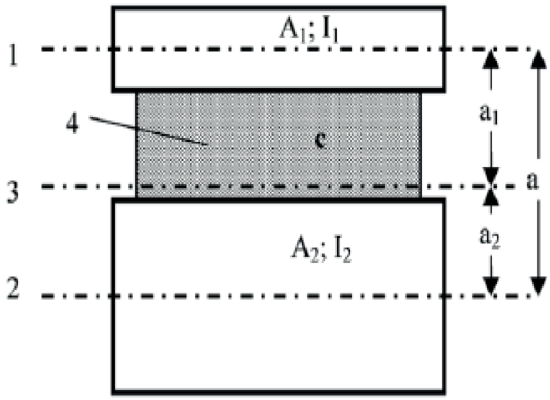
## 3. PROFİL KARAKTERİSLİKLERİN HESABI

Hesaplarda kullanılacak olan Atalet momenti ve mukavemet momenti değerleri NF EN 14024 şartnamesi Metod III kullanılarak yapılmıştır.

- 1 Centre de gravité de la section métallique 1 avec une surface *A1* et un moment d'inertie *I1*
- 2 Centre de gravité de la section métallique 2 avec une surface *A2* et un moment d'inertie *I2*
- 3 Centre de gravité du profilé composite
- 4 Coupure thermique avec constante d'élasticité *c*



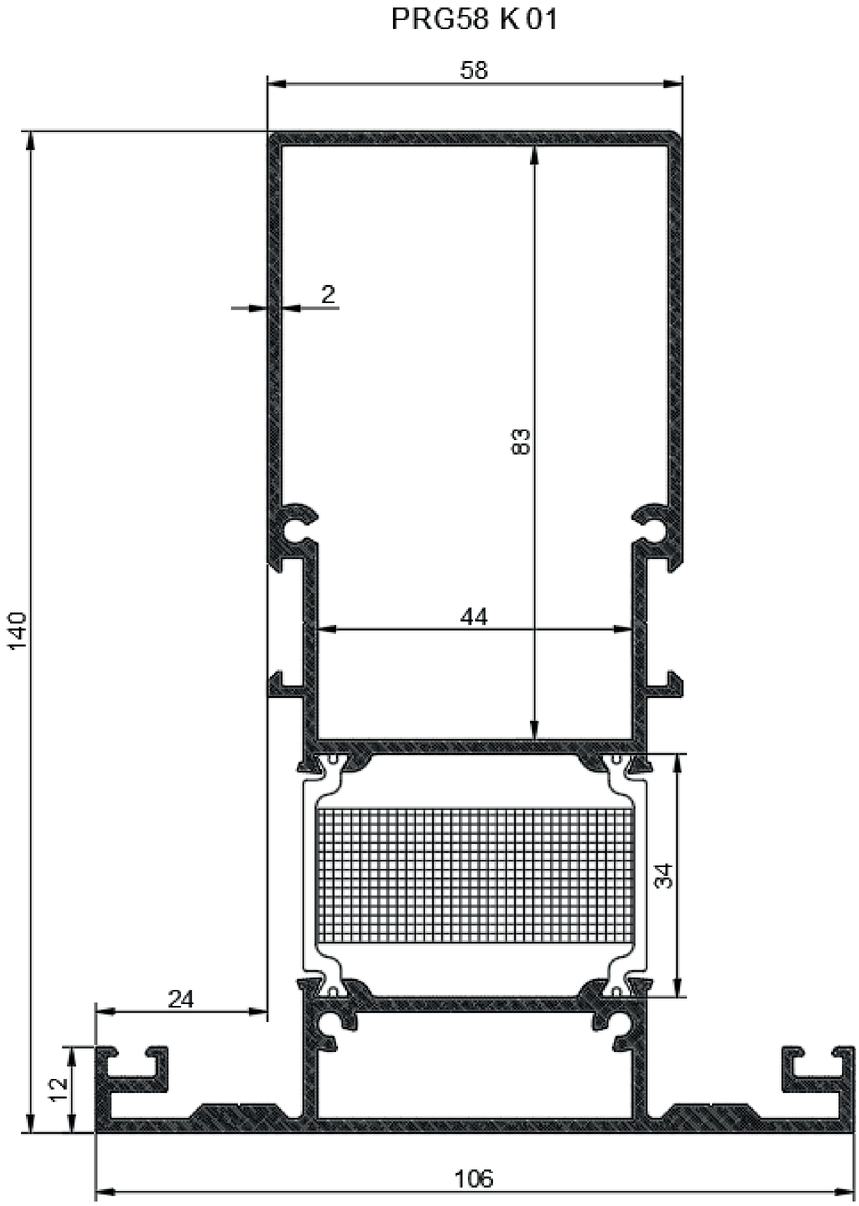

Şekil 6 Bir RPT Profilinin Şematik Gösterimi

Bu basitleştirilmiş hesaplama yöntemi, düz veya açılı şekil şeritleriyle (boru şeklinde olmayan) ve minimum 1,5 mm et kalınlığıyla sınırlıdır.

Mesnetler arasındaki serbest açıklığın bir fonksiyonu olarak etkin atalet (Ief), aşağıdaki tablodaki formüllerle hesaplanır.

| **Portée libre entre appuis ($\ell$) en mm** | **Formule de calculs de *Ief en mm*$^4$** |
|---|---|
| $\ell \leq 1000$ | $Ief = I_1+I_2$ |
| $1000 \leq \ell \leq 6000$ | $Ief = (0,9Is-(I_1+I_2))(\ell/1000 -1)/5 + (I_1+I_2)$ |
| $\ell \geq 6000$ | $Ief = 0,9Is$ |

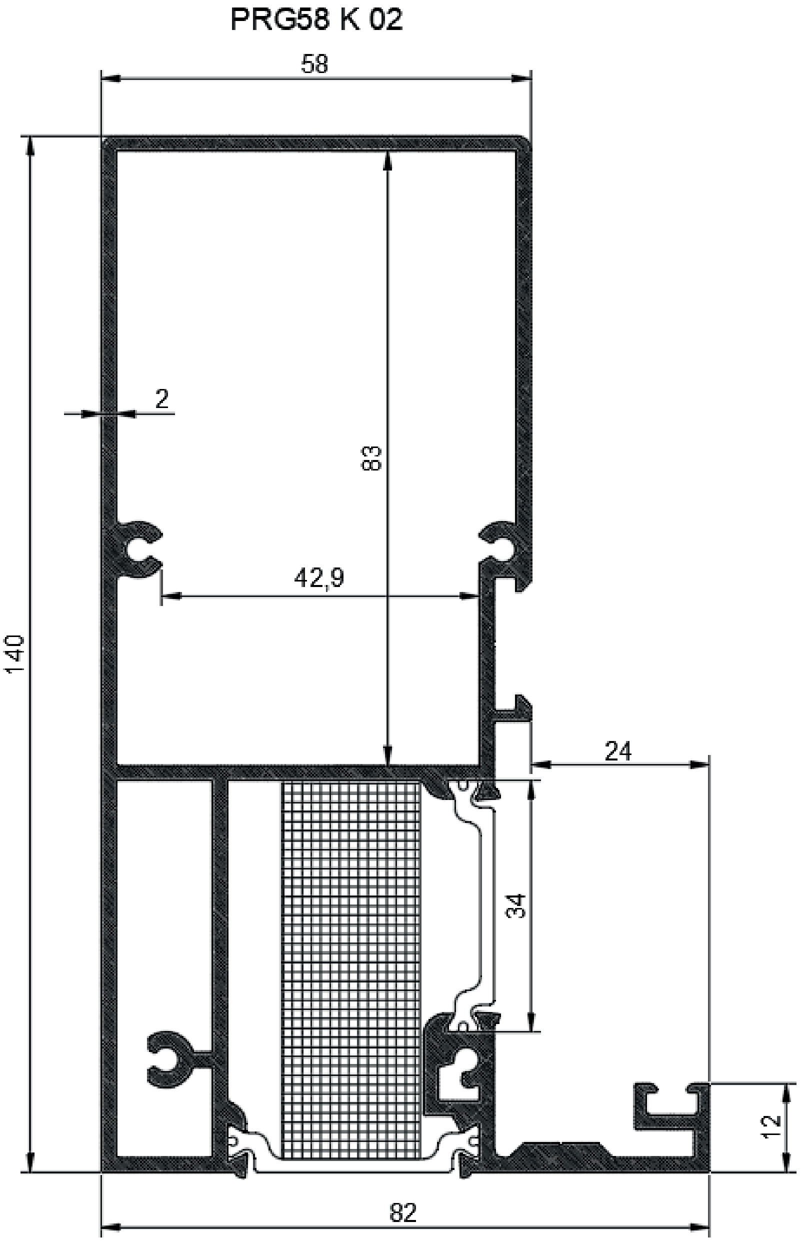
## 3.1 Orta Aşık Profili




Şekil 7 Aşık Profili Kesiti

| | |
|---|---|
| Atalet Momenti Ixx: | 2159800 $mm^4$ |
| Atalet Momenti Iyy: | 867830 $mm^4$ |
| Mukavemet Momenti Wxx: | 31130 $mm^3$ |
| Mukavemet Momenti Wyy: | 16408 $mm^3$ |

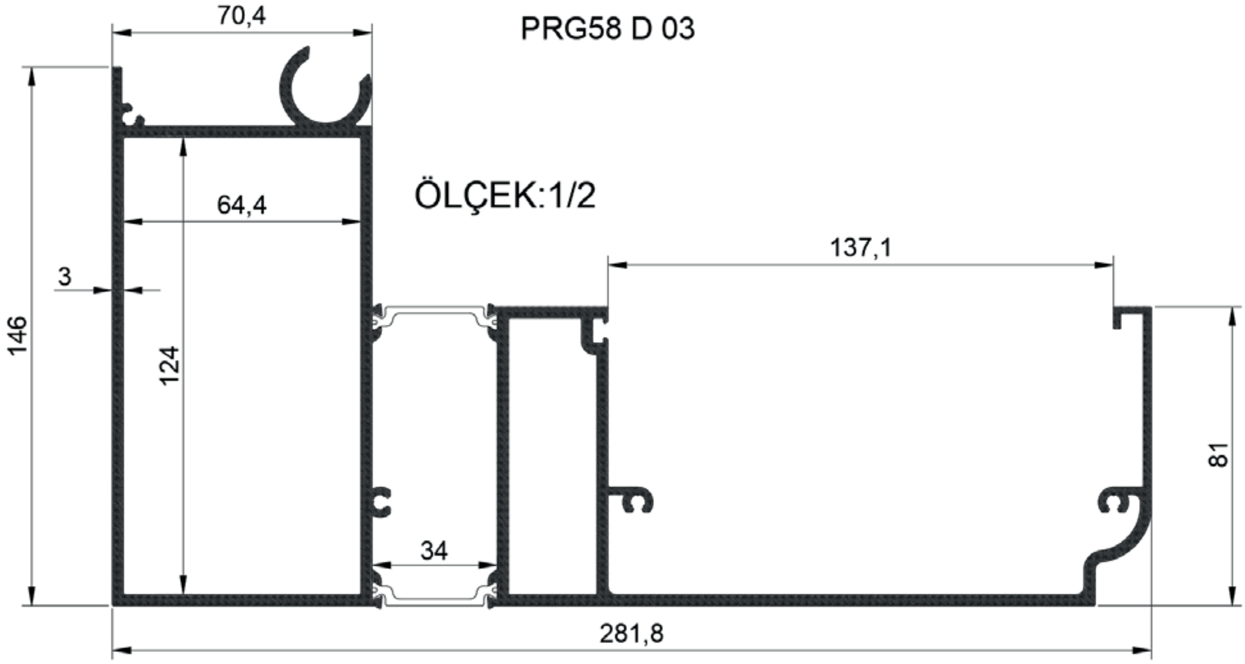
## 3.2 Kenar Aşık Profili




Şekil 8 Aşık Profili Kesiti

| | |
|---|---|
| Atalet Momenti Ixx: | 1902890 $mm^4$ |
| Atalet Momenti Iyy: | 739560 $mm^4$ |
| Mukavemet Momenti Wxx: | 32575 $mm^3$ |
| Mukavemet Momenti Wyy: | 24157 $mm^3$ |

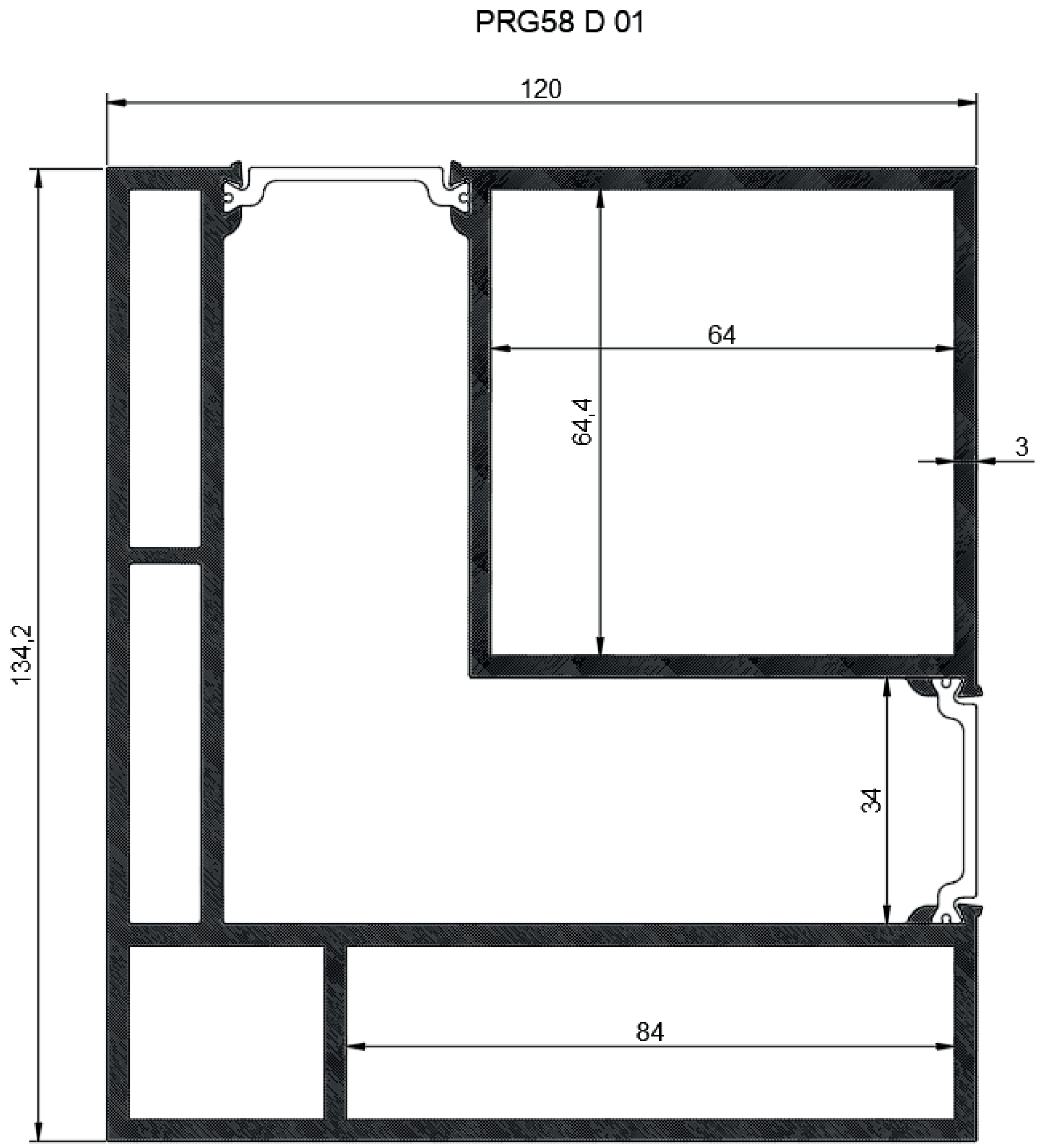
## 3.3 Kiriş ve Dere Profili




Şekil 9 Kiriş ve Dere Profili Kesiti

| | |
|---|---|
| Atalet Momenti Ixx: | 6085690 $mm^4$ |
| Atalet Momenti Iyy: | 15332050 $mm^4$ |
| Mukavemet Momenti Wxx: | 120484 $mm^3$ |
| Mukavemet Momenti Wyy: | 146256 $mm^3$ |

## 3.4 Kolon Profili




Şekil 10 Kolon Profili Kesiti

| | |
|---|---|
| Atalet Momenti Ixx: | 4379410 $mm^4$ |
| Atalet Momenti Iyy: | 5049500 $mm^4$ |
| Mukavemet Momenti Wxx: | 67687 $mm^3$ |
| Mukavemet Momenti Wyy: | 91926 $mm^3$ |

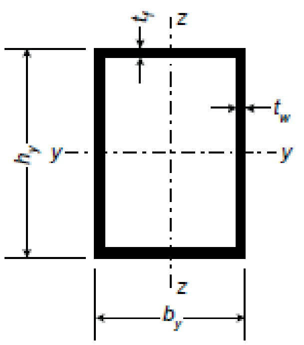
# 4. GERİLME KONTROLLERİ

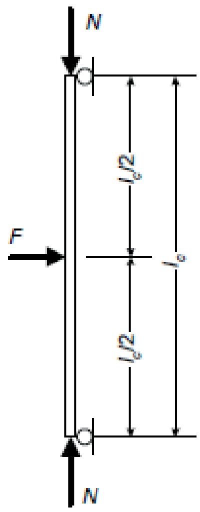
## 4.1 Aşık Hesabı

Maksimum Moment: 2,46 kNm

| Ec9_ex92 | Axial force and bending moment | Page 1 of 5 |
|---|---|---|

### Example 9.2 Beam-column with rectangular hollow section




### Dimensions and material properties

$MPa \equiv 10^6 \cdot Pa$

$kN \equiv 1000 \cdot newton$

Length $\quad l_c := 3030 \cdot mm$

Thickness $\quad t_w := 2 \cdot mm \qquad t_f := 3 \cdot mm$

Width $\quad h_y := 135 \cdot mm \qquad h_i := h_y - 2 \cdot t_f \qquad h_i = 129 \cdot mm$

$b_y := 53 \cdot mm \qquad b_i := b_y - 2 \cdot t_w \qquad b_i = 49 \cdot mm$

[1] Table 3.2b $\quad$ Alloy: EN AW-6060 T66 EP $t < 15$ mm $\qquad f_o := 160 \cdot MPa \qquad f_u := 215 \cdot MPa$

$BC := \text{"A"} \qquad E := 70000 \cdot MPa \qquad \gamma_{M1} := 1.1 \qquad \gamma_{M2} := 1.25$

### Forces and moment

Axial force $\quad N_{Ed} := 0 \cdot kN$

Transverse force $\quad F_{Ed} := 3.25 \cdot kN$

Bending moment $\quad M_{y.Ed} := \dfrac{F_{Ed} \cdot l_c}{4} \qquad M_{y.Ed} = 2.462 \cdot kN \cdot m$

### Classification of the cross section in axial compression

Web $\quad \beta_w := \dfrac{h_y - 2 \cdot t_f}{t_w} \qquad \beta_w = 64.5 \qquad \varepsilon := \sqrt{\dfrac{250 \cdot MPa}{f_o}} \qquad \varepsilon = 1.25$

[1] Tab. 6.2 $\quad \beta_{1w} := 11 \cdot \varepsilon \qquad \beta_{2w} := 16 \cdot \varepsilon \qquad \beta_{3w} := 22 \cdot \varepsilon$

$class_c := if(\beta_w > \beta_{1w}, if(\beta_w > \beta_{2w}, if(\beta_w > \beta_{3w}, 4, 3), 2), 1) \qquad class_c = 4$

[1] 6.3.1 $\quad A_e := A \qquad \eta := 1.0$

### Classification of the cross section in *y-y* axis bending

Web $\quad \beta_w := 0.4 \cdot \dfrac{h_y - 2 \cdot t_f}{t_w} \qquad \beta_w = 25.8$

| T Höglund | aluMATTER | 29.01.2021 |
|---|---|---|

[1] Tab. 5.1 $\beta_{1w} := 11 \cdot \varepsilon$ $\beta_{2w} := 16 \cdot \varepsilon$ $\beta_{3w} := 22 \cdot \varepsilon$

[1] 5.4.5 $class_w := if(\beta_w > \beta_{1w}, if(\beta_w > \beta_{2w}, if(\beta_w > \beta_{3w}, 4, 3), 2), 1)$ $class_w = 3$

Flange $\beta_f := \dfrac{b_y - 2 \cdot t_w}{t_f}$ $\beta_f = 16.333$

[1] Tab. 5.1 $\beta_{1f} := 11 \cdot \varepsilon$ $\beta_{2f} := 16 \cdot \varepsilon$ $\beta_{3f} := 22 \cdot \varepsilon$

[1] 5.4.5 $class_f := if(\beta_f > \beta_{1w}, if(\beta_f > \beta_{2f}, if(\beta_f > \beta_{3f}, 4, 3), 2), 1)$ $class_f = 2$

Classification of the total cross-section in *y-y* axis bending:

$class_y := if(class_f > class_w, class_f, class_w)$ $class_y = 3$

## Classification of the cross section in z-z axis bending

Web $\beta_w := 0.4 \cdot \dfrac{b_y - 2 \cdot t_w}{t_f}$ $\beta_w = 6.533$

[1] Tab. 5.1 $\beta_{1w} := 11 \cdot \varepsilon$ $\beta_{2w} := 16 \cdot \varepsilon$ $\beta_{3w} := 22 \cdot \varepsilon$

[1] 5.4.5 $class_w := if(\beta_w > \beta_{1w}, if(\beta_w > \beta_{2w}, if(\beta_w > \beta_{3w}, 4, 3), 2), 1)$ $class_w = 1$

Flange $\beta_f := \dfrac{h_y - 2 \cdot t_f}{t_w}$ $\beta_f = 64.5$

[1] Tab. 5.1 $\beta_{1f} := 11 \cdot \varepsilon$ $\beta_{2f} := 16 \cdot \varepsilon$ $\beta_{3f} := 22 \cdot \varepsilon$

[1] 5.4.5 $class_f := if(\beta_f > \beta_{1w}, if(\beta_f > \beta_{2f}, if(\beta_f > \beta_{3f}, 4, 3), 2), 1)$ $class_f = 4$

Classification of the total cross-section in *z-z* axis bending:

$class_z := if(class_f > class_w, class_f, class_w)$ $class_z = 4$

## Cross section constants

$A := b_y \cdot h_y - b_i \cdot h_i$ $A = 834 \cdot mm^2$

$I_y := \dfrac{b_y \cdot h_y^3}{12} - \dfrac{b_i \cdot h_i^3}{12}$ $I_y = 2.101 \times 10^6 \cdot mm^4$ $I_z := \dfrac{h_y \cdot b_y^3}{12} - \dfrac{h_i \cdot b_i^3}{12}$ $I_z = 4.1 \times 10^5 \cdot mm^4$

$W_{el.y} := \dfrac{I_y \cdot 2}{h_y}$ $W_{el.y} = 3.113 \times 10^4 \cdot mm^3$ $W_{el.z} := \dfrac{I_z \cdot 2}{b_y}$ $W_{el.z} = 1.5 \times 10^4 \cdot mm^3$

$W_{pl.y} := \dfrac{b_y \cdot h_y^2}{4} - \dfrac{b_i \cdot h_i^2}{4}$ $W_{pl.y} = 3.763 \times 10^4 \cdot mm^3$

[1] 5.6.2.1 $class_y = 3$ $\alpha_y := \dfrac{W_{pl.y}}{W_{el.y}}$ $\alpha_y = 1.209$ $class_z = 4$ $\alpha_z := 1$

$i_y := \sqrt{\dfrac{I_y}{A}}$ $i_y = 50.2 \cdot mm$ $i_z := \sqrt{\dfrac{I_z}{A}}$ $i_z = 22.2 \cdot mm$

## Flexural buckling

TALAT (5.6) $\lambda_y := \frac{l_c}{\pi \cdot i_y} \cdot \sqrt{\frac{\eta \cdot f_o}{E}}$ $\lambda_y = 1.07$ $\lambda_z := \frac{l_c}{\pi \cdot i_z} \cdot \sqrt{\frac{\eta \cdot f_o}{E}}$ $\lambda_z = 2.422$

[1] 5.8.4.1 $\phi_y := 0.5 \cdot \left[1 + \left[0.20 \cdot (\lambda_y - 0.1) + \lambda_y^2\right]\right]$ $\phi_z := 0.5 \cdot \left[1 + \left[0.20 \cdot (\lambda_z - 0.1) + \lambda_z^2\right]\right]$

$\chi_y := \frac{1}{\phi_y + \sqrt{\phi_y^2 - \lambda_y^2}}$ $\chi_y = 0.609$ $\chi_z := \frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_z^2}}$ $\chi_z = 0.156$

$N_{Rd} := \frac{A \cdot f_o}{\gamma_{M1}}$ $N_{Rd} = 121.3 \cdot kN$

Exponents in interaction formula

[1] 5.9.4.2 (4) $\psi := \alpha_z \cdot \alpha_y$ $\psi := if(\psi > 2, 2, \psi)$ $\psi = 1.209$

$\psi_c := \chi_z \cdot \psi$ $\psi_c := if(\psi_c < 0.8, 0.8, \psi_c)$ $\psi_c = 0.8$

Two checks should in principle be made:
1. As if there were no welds
2. Check in the section of the weld

In this example it is obvious that the section with the weld is designing. However, to illustrate the procedures both checks are made.

### Mid section or the member without weld

[1] 5.9.4.5 $\omega_o := 1$ $\omega_o = 1$ $\omega_x := \omega_o$

$M_{y.Rd} := \alpha_y \cdot W_{el.y} \cdot \frac{f_o}{\gamma_{M1}}$ $M_{y.Rd} = 5.473 \cdot kN \cdot m$ $M_{y.Ed} = 3.354 \cdot kN \cdot m$

$M_{z.Rd} := \alpha_z \cdot W_{el.z} \cdot \frac{f_o}{\gamma_{M1}}$ $M_{z.Rd} = 2.251 \cdot kN \cdot m$ $M_{z.Ed} := 0 \cdot kN \cdot m$

$\chi_{min} := \chi_y$ $\chi_{min} = 0.609$

### Flexural buckling check

[1] 5.9.4.2 (4) $\left(\frac{N_{Ed}}{\chi_{min} \cdot \omega_x \cdot N_{Rd}}\right)^{\psi_c} + \frac{1}{\omega_o} \cdot \left[\left(\frac{M_{y.Ed}}{M_{y.Rd}}\right)^{1.7} + \left(\frac{M_{z.Ed}}{M_{z.Rd}}\right)^{1.7}\right]^{0.6} = 0.470$ < 1,0 OK !

### Cross weld in mid section

[1] Tab. 5.2 HAZ softening factor $\rho_{haz} := 0.65$

[1] 5.9.4.5 $\omega_o := \frac{\rho_{haz} \cdot f_u \cdot \gamma_{M1}}{f_o \cdot \gamma_{M2}}$ $\omega_o = 0.769$ $\omega_x := \omega_o$

$M_{y.Rd} := \alpha_y \cdot W_{el.y} \cdot \frac{f_o}{\gamma_{M1}}$ $M_{y.Rd} = 5.473 \cdot kN \cdot m$ $M_{y.Ed} = 2.462 \cdot kN \cdot m$

| Ec9_ex92 | Axial force and bending moment | Page 4 of 5 |
|---|---|---|

$$M_{z.Rd} := \alpha_z \cdot W_{el.z} \cdot \frac{f_o}{\gamma_{M1}} \qquad M_{z.Rd} = 2.251 \cdot kN \cdot m \qquad M_{z.Ed} := 0 \cdot kN \cdot m$$

[1] (6.68a) $\lambda_{y.haz} := \lambda_y \sqrt{\omega_o}$ $\quad \lambda_{y.haz} = 0.938 \quad$ $\lambda_{z.haz} := \lambda_z \sqrt{\omega_o}$ $\quad \lambda_{z.haz} = 2.124$

[1] 5.8.4.1 $\phi_y := 0.5 \cdot \left[1 + \left[0.20 \cdot (\lambda_{y.haz} - 0.1) + \lambda_{y.haz}^2\right]\right]$ $\quad \phi_z := 0.5 \cdot \left[1 + \left[0.20 \cdot (\lambda_{z.haz} - 0.1) + \lambda_{z.haz}^2\right]\right]$

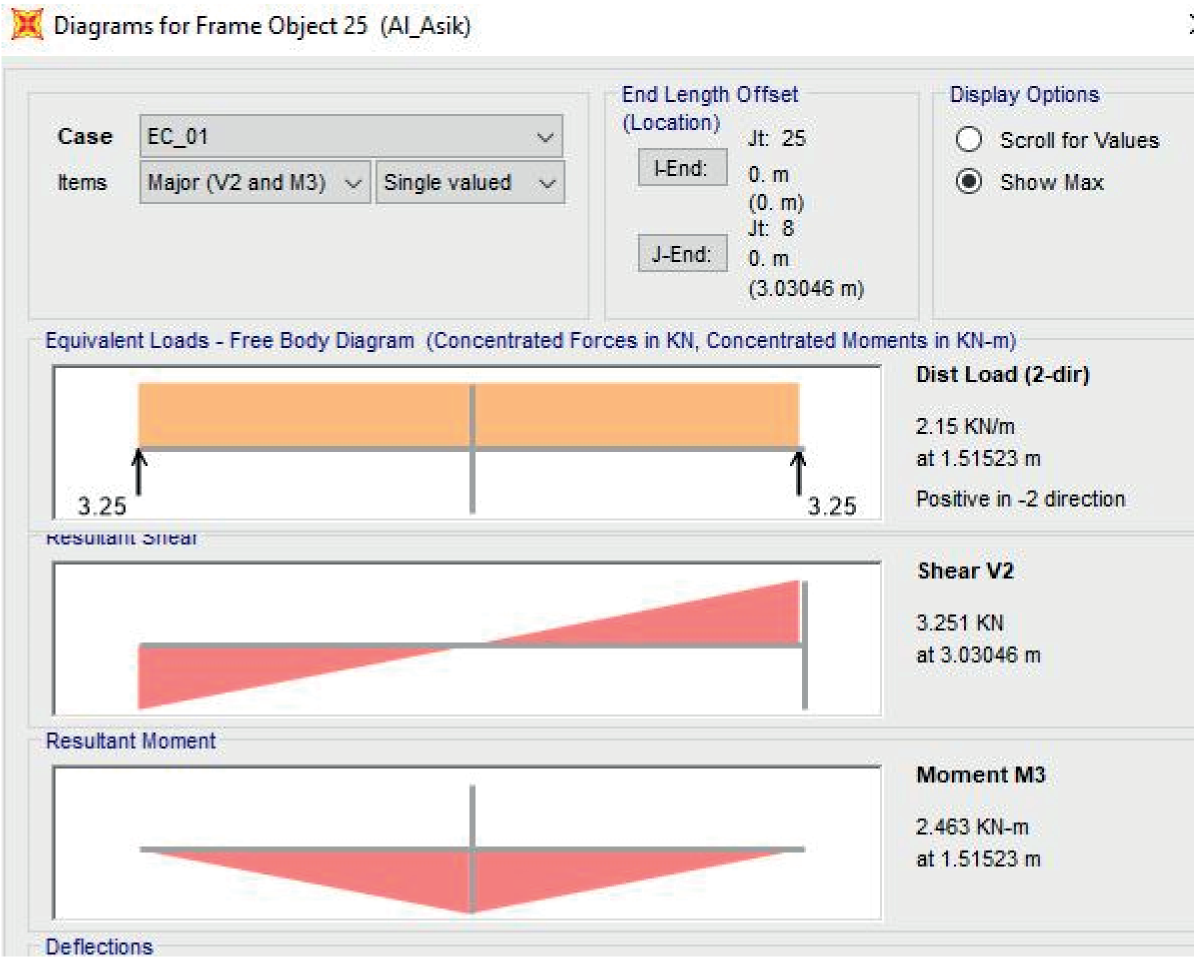
$$\chi_y := \frac{1}{\phi_y + \sqrt{\phi_y^2 - \lambda_{y.haz}^2}} \qquad \chi_y = 0.697 \qquad \chi_z := \frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_{z.haz}^2}} \qquad \chi_z = 0.199$$

$$N_{Rd} := \frac{A \cdot f_o}{\gamma_{M1}} \qquad N_{Rd} = 121.3 \cdot kN$$

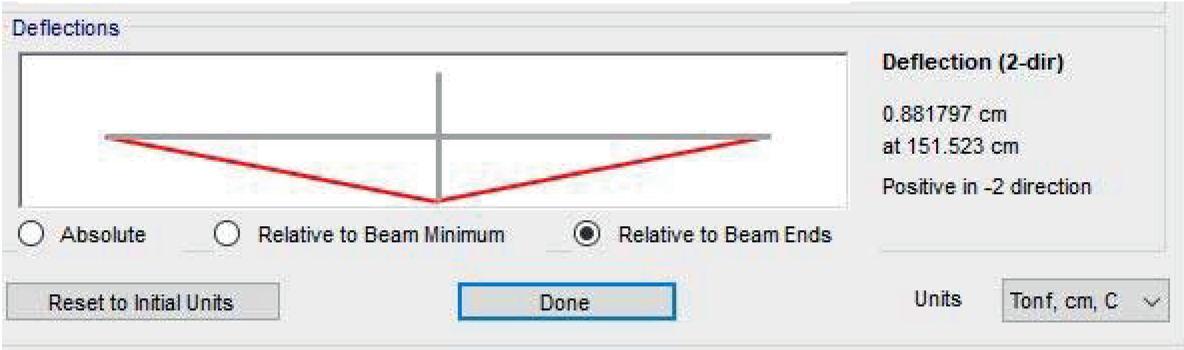
$$\chi_{min} := \chi_y \qquad \chi_{min} = 0.697$$

## Flexural buckling check in section with weld

[1] 5.9.4.2 (4) $\left(\frac{N_{Ed}}{\chi_{min} \cdot \omega_x \cdot N_{Rd}}\right)^{\psi_c} + \frac{1}{\omega_o} \cdot \left[\left(\frac{M_{y.Ed}}{M_{y.Rd}}\right)^{1.7} + \left(\frac{M_{z.Ed}}{M_{z.Rd}}\right)^{1.7}\right]^{0.6} = 0.684$ $\quad$ < 1,0 OK !




Şekil 11 Orta Aşık Moment Diyagramı

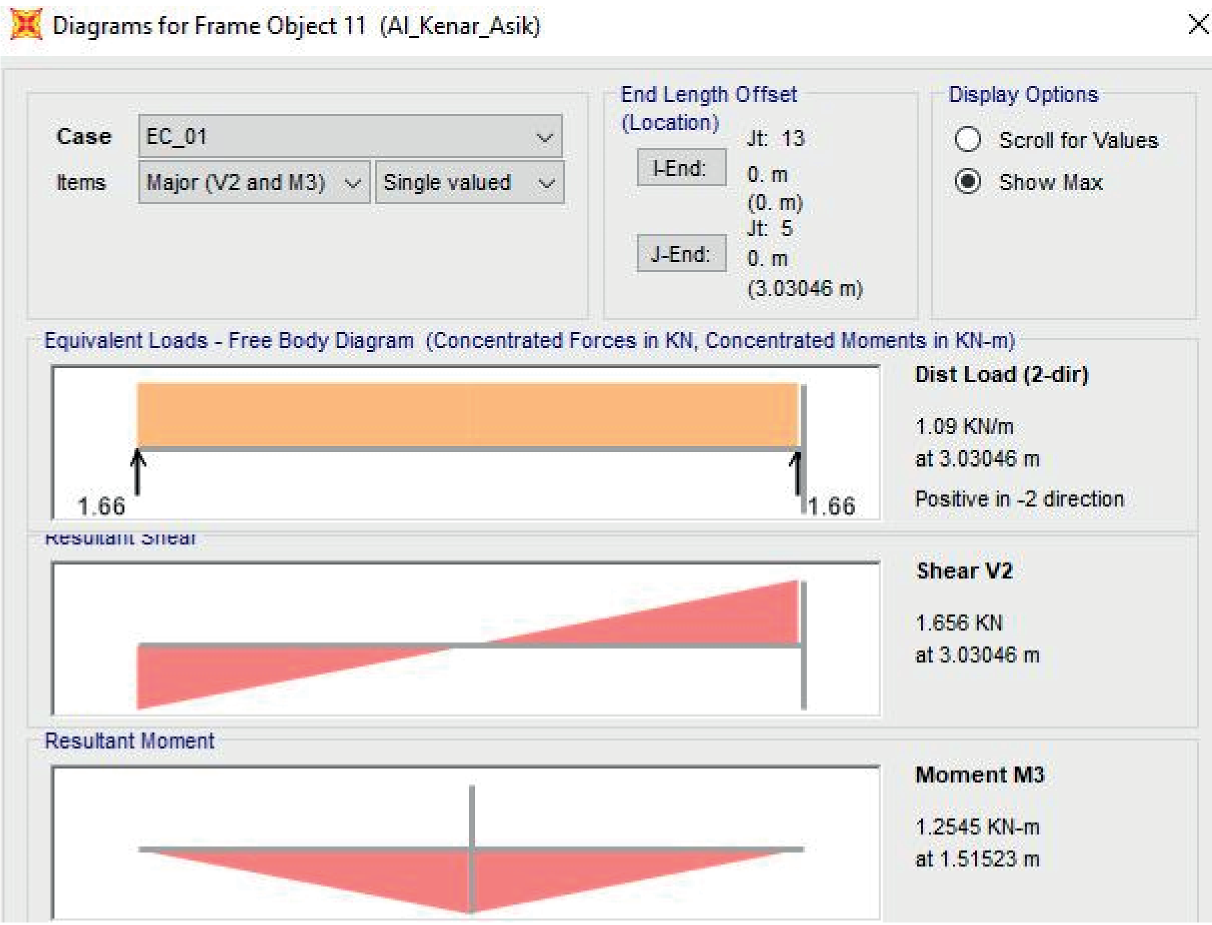



Maks. Düşey Deplasman =0,88 cm > L/300 =300/300 = 1,00 cm ***Uygundur.***

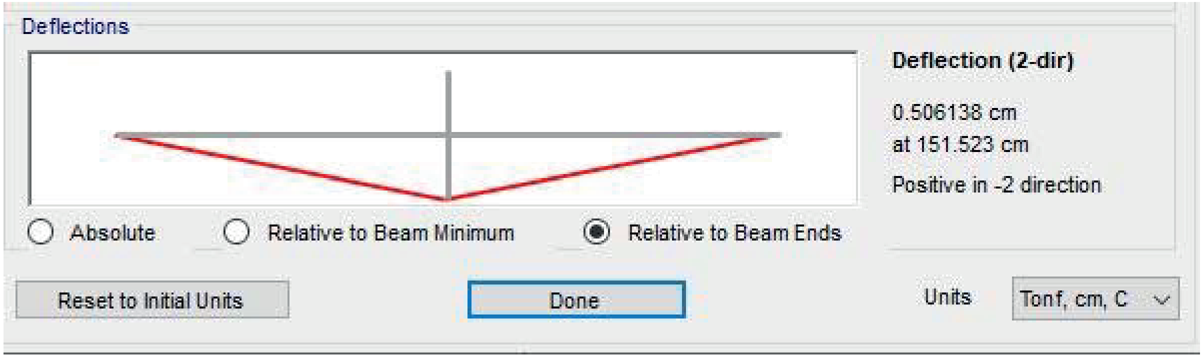
Aşık profili 3.00 m açıklıkta gerekli dayanım ve sehim değerlerini sağlamaktadır.

## 4.2 Kenar Aşık Hesabı

Maksimum Moment: 1,25 kNm




Şekil 12 Kenar Aşık Moment Diyagramı

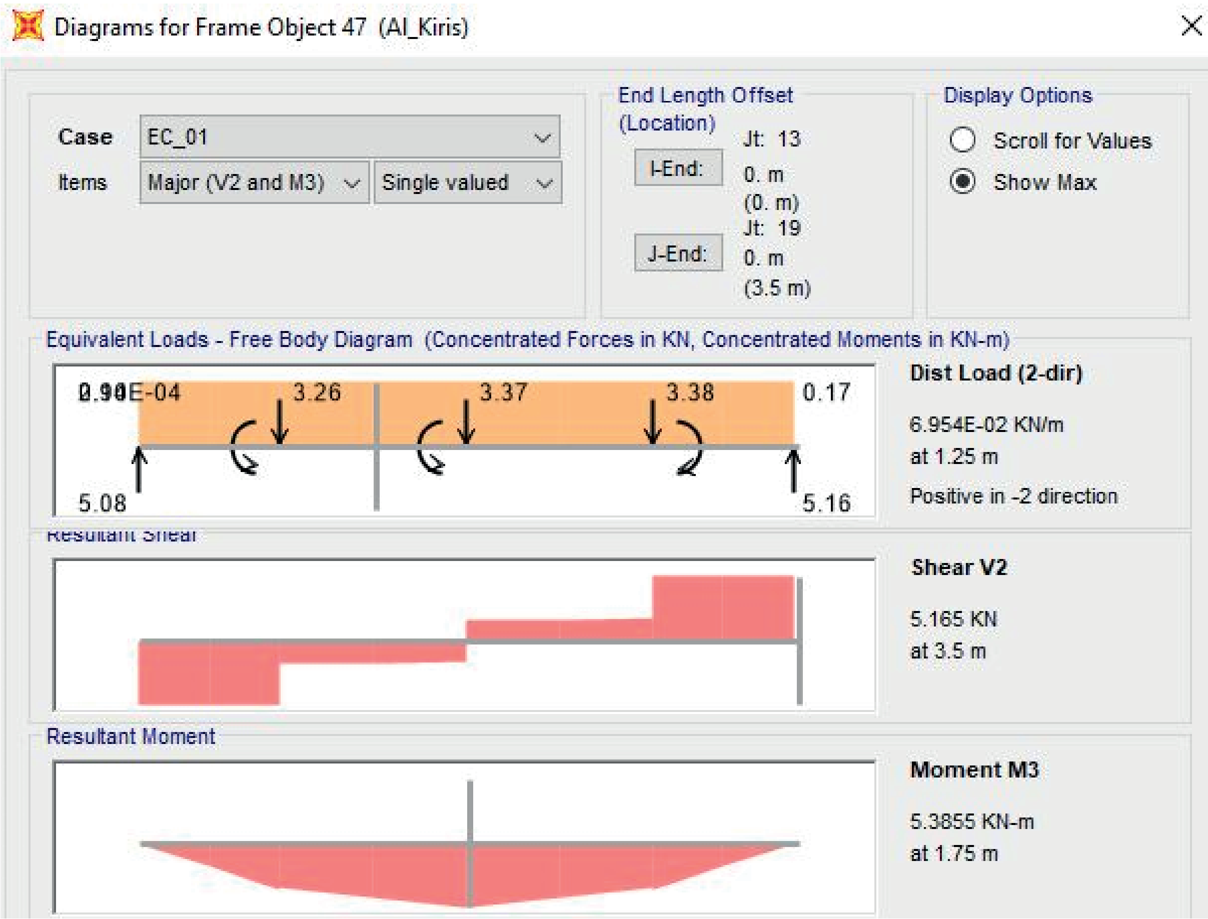



Maks. Düşey Deplasman =0,51 cm > L/300 =300/300 = 1,00 cm ***Uygundur.***

Kenar Aşık profili 3.00 m açıklıkta gerekli dayanım ve sehim değerlerini sağlamaktadır.

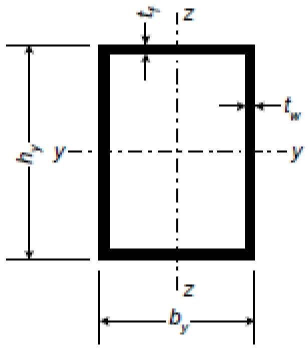
## 4.3 Kiriş Hesabı

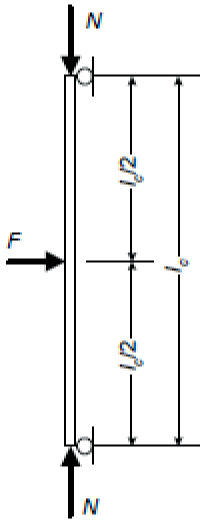
Maksimum Moment: 5,39 kNm




Şekil 13 Ana Kiriş Moment Diyagramı

# Example 9.2 Beam-column with rectangular hollow section




## Dimensions and material properties

$MPa \equiv 10^6 \cdot Pa$

$kN \equiv 1000 \cdot newton$

Length $l_c := 3500 \cdot mm$

Thickness $t_w := 3.7 \cdot mm$ $\quad t_f := 3.7 \cdot mm$

Width $h_y := 130 \cdot mm$ $\quad h_i := h_y - 2 \cdot t_f$ $\quad h_i = 122.6 \cdot mm$

$b_y := 70 \cdot mm$ $\quad b_i := b_y - 2 \cdot t_w$ $\quad b_i = 62.6 \cdot mm$

[1] Table 3.2b Alloy: EN AW-6060 T66 EP $t < 15$ mm $\quad f_o := 160 \cdot MPa$ $\quad f_u := 215 \cdot MPa$

$BC := \text{"A"}$ $\quad E := 70000 \cdot MPa$ $\quad \gamma_{M1} := 1.1$ $\quad \gamma_{M2} := 1.25$

## Forces and moment

Axial force $N_{Ed} := 0 \cdot kN$

Transverse force $F_{Ed} := 6.16 \cdot kN$

Bending moment $M_{y.Ed} := \dfrac{F_{Ed} \cdot l_c}{4}$ $\quad M_{y.Ed} = 5.39 \cdot kN/m$

## Classification of the cross section in axial compression

Web $\beta_w := \dfrac{h_y - 2 \cdot t_f}{t_w}$ $\quad \beta_w = 33.135$ $\quad \varepsilon := \sqrt{\dfrac{250 \cdot MPa}{f_o}}$ $\quad \varepsilon = 1.25$

[1] Tab. 6.2 $\beta_{1w} := 11 \cdot \varepsilon$ $\quad \beta_{2w} := 16 \cdot \varepsilon$ $\quad \beta_{3w} := 22 \cdot \varepsilon$

$class_c := if(\beta_w > \beta_{1w}, if(\beta_w > \beta_{2w}, if(\beta_w > \beta_{3w}, 4, 3), 2), 1)$ $\quad class_c = 4$

[1] 6.3.1 $A_e := A$ $\quad \eta := 1.0$

## Classification of the cross section in *y-y* axis bending

Web $\beta_w := 0.4 \cdot \dfrac{h_y - 2 \cdot t_f}{t_w}$ $\quad \beta_w = 13.254$

[1] Tab. 5.1 $\beta_{1w} := 11\cdot\varepsilon$ $\beta_{2w} := 16\cdot\varepsilon$ $\beta_{3w} := 22\cdot\varepsilon$

[1] 5.4.5 $class_w := if(\beta_w > \beta_{1w}, if(\beta_w > \beta_{2w}, if(\beta_w > \beta_{3w}, 4, 3), 2), 1)$ $class_w = 1$

Flange $\beta_f := \dfrac{b_y - 2\cdot t_w}{t_f}$ $\beta_f = 16.919$

[1] Tab. 5.1 $\beta_{1f} := 11\cdot\varepsilon$ $\beta_{2f} := 16\cdot\varepsilon$ $\beta_{3f} := 22\cdot\varepsilon$

[1] 5.4.5 $class_f := if(\beta_f > \beta_{1w}, if(\beta_f > \beta_{2f}, if(\beta_f > \beta_{3f}, 4, 3), 2), 1)$ $class_f = 2$

Classification of the total cross-section in *y-y* axis bending:

$class_y := if(class_f > class_w, class_f, class_w)$ $class_y = 2$

## Classification of the cross section in z-z axis bending

Web $\beta_w := 0.4\cdot\dfrac{b_y - 2\cdot t_w}{t_f}$ $\beta_w = 6.768$

[1] Tab. 5.1 $\beta_{1w} := 11\cdot\varepsilon$ $\beta_{2w} := 16\cdot\varepsilon$ $\beta_{3w} := 22\cdot\varepsilon$

[1] 5.4.5 $class_w := if(\beta_w > \beta_{1w}, if(\beta_w > \beta_{2w}, if(\beta_w > \beta_{3w}, 4, 3), 2), 1)$ $class_w = 1$

Flange $\beta_f := \dfrac{h_y - 2\cdot t_f}{t_w}$ $\beta_f = 33.135$

[1] Tab. 5.1 $\beta_{1f} := 11\cdot\varepsilon$ $\beta_{2f} := 16\cdot\varepsilon$ $\beta_{3f} := 22\cdot\varepsilon$

[1] 5.4.5 $class_f := if(\beta_f > \beta_{1w}, if(\beta_f > \beta_{2f}, if(\beta_f > \beta_{3f}, 4, 3), 2), 1)$ $class_f = 4$

Classification of the total cross-section in *z-z* axis bending:

$class_z := if(class_f > class_w, class_f, class_w)$ $class_z = 4$

## Cross section constants

$A := b_y\cdot h_y - b_i\cdot h_i$ $A = 1.425\times 10^3\cdot mm^2$

$I_y := \dfrac{b_y\cdot h_y^3}{12} - \dfrac{b_i\cdot h_i^3}{12}$ $I_y = 3.203\times 10^6\cdot mm^4$ $I_z := \dfrac{h_y\cdot b_y^3}{12} - \dfrac{h_i\cdot b_i^3}{12}$ $I_z = 1.21\times 10^6\cdot mm^4$

$W_{el.y} := \dfrac{I_y\cdot 2}{h_y}$ $W_{el.y} = 4.927\times 10^4\cdot mm^3$ $W_{el.z} := \dfrac{I_z\cdot 2}{b_y}$ $W_{el.z} = 3.5\times 10^4\cdot mm^3$

$W_{pl.y} := \dfrac{b_y\cdot h_y^2}{4} - \dfrac{b_i\cdot h_i^2}{4}$ $W_{pl.y} = 6.052\times 10^4\cdot mm^3$

[1] 5.6.2.1 $class_y = 2$ $\alpha_y := \dfrac{W_{pl.y}}{W_{el.y}}$ $\alpha_y = 1.228$ $class_z = 4$ $\alpha_z := 1$

$i_y := \sqrt{\dfrac{I_y}{A}}$ $i_y = 47.4\cdot mm$ $i_z := \sqrt{\dfrac{I_z}{A}}$ $i_z = 29.1\cdot mm$

## Flexural buckling

TALAT (5.6) $\lambda_y := \frac{l_c}{\pi \cdot i_y} \cdot \sqrt{\frac{\eta \cdot f_o}{E}}$ $\lambda_y = 1.284$ $\lambda_z := \frac{l_c}{\pi \cdot i_z} \cdot \sqrt{\frac{\eta \cdot f_o}{E}}$ $\lambda_z = 2.09$

[1] 5.8.4.1 $\phi_y := 0.5 \cdot \left[1 + \left[0.20 \cdot (\lambda_y - 0.1) + \lambda_y^2\right]\right]$ $\phi_z := 0.5 \cdot \left[1 + \left[0.20 \cdot (\lambda_z - 0.1) + \lambda_z^2\right]\right]$

$\chi_y := \frac{1}{\phi_y + \sqrt{\phi_y^2 - \lambda_y^2}}$ $\chi_y = 0.476$ $\chi_z := \frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_z^2}}$ $\chi_z = 0.205$

$N_{Rd} := \frac{A \cdot f_o}{\gamma_{M1}}$ $N_{Rd} = 207.3 \cdot kN$

Exponents in interaction formula

[1] 5.9.4.2 (4) $\psi := \alpha_z \cdot \alpha_y$ $\psi := if(\psi > 2, 2, \psi)$ $\psi = 1.228$

$\psi_c := \chi_z \cdot \psi$ $\psi_c := if(\psi_c < 0.8, 0.8, \psi_c)$ $\psi_c = 0.8$

Two checks should in principle be made:
1. As if there were no welds
2. Check in the section of the weld

In this example it is obvious that the section with the weld is designing. However, to illustrate the procedures both checks are made.

### Mid section or the member without weld

[1] 5.9.4.5 $\omega_o := 1$ $\omega_o = 1$ $\omega_x := \omega_o$

$M_{y.Rd} := \alpha_y \cdot W_{el.y} \cdot \frac{f_o}{\gamma_{M1}}$ $M_{y.Rd} = 8.803 \cdot kN \cdot m$ $M_{y.Ed} = 7 \cdot kN \cdot m$

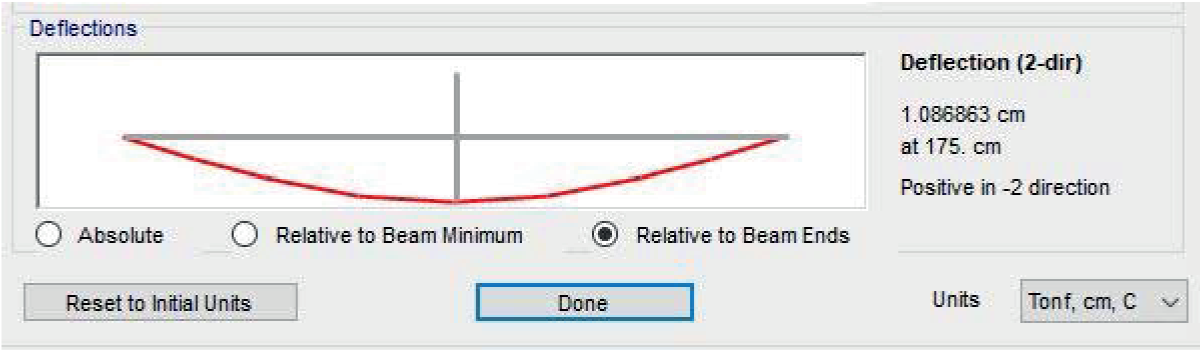
$M_{z.Rd} := \alpha_z \cdot W_{el.z} \cdot \frac{f_o}{\gamma_{M1}}$ $M_{z.Rd} = 5.027 \cdot kN \cdot m$ $M_{z.Ed} := 0 \cdot kN \cdot m$

$\chi_{min} := \chi_y$ $\chi_{min} = 0.476$

### Flexural buckling check

[1] 5.9.4.2 (4) $\left(\frac{N_{Ed}}{\chi_{min} \cdot \omega_x \cdot N_{Rd}}\right)^{\psi_c} + \frac{1}{\omega_o} \cdot \left[\left(\frac{M_{y.Ed}}{M_{y.Rd}}\right)^{1.7} + \left(\frac{M_{z.Ed}}{M_{z.Rd}}\right)^{1.7}\right]^{0.6} = 0.650$ < 1,0 OK !

ALUSKY




Maks. Düşey Deplasman =1,09 cm >  L/300 =350/300 = 1,167 cm   ***Uygundur.***

Kolonlar arasındaki kiriş profili 3.50 m açıklıkta gerekli dayanım ve sehim değerlerini sağlamaktadır.

## 4.4 Kolon Hesabı

Maksimum Eksenel Yük: 6,94 kN

Kolon Eksenel Yük Kapasitesi: 49,20 kN

Ec9_ex51_Compression_RHS.xmcd Axial force resistance Page 1 of 1

### Example 5.1. Axial force resistance of member with square hollow section

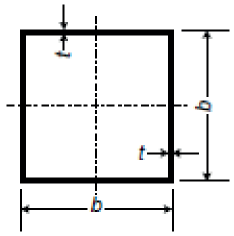
#### Dimensions and material properties

| | | | |
|---|---|---|---|
| Cross section width | $b := 70 \cdot mm$ | $MPa \equiv 10^6 \cdot Pa$ | $kN \equiv 1000 \cdot newton$ |
| Thickness | $t := 3 \cdot mm$ | 6060-T66, $t < 10$ | $f_o := 160 \cdot MPa$ |
| Column length | $L := 2500 \cdot mm$ | $E := 70000 \cdot MPa$ | $\gamma_{M1} := 1.1$ |

6.1.4.3 **Slenderness parameter**

$b_f := b - 2 \cdot t$ $\quad b_f = 64 mm$

$\beta := \frac{b_f}{t}$ $\quad \beta = 21.3$




6.1.4.4 **Element classification**

$\varepsilon := \sqrt{\frac{250 \cdot MPa}{f_o}}$ $\quad \varepsilon = 1.25$

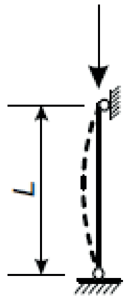
Table 6.2

$\beta_1 := 11 \cdot \varepsilon$ $\quad \beta_1 = 13.8$

$\beta_2 := 16 \cdot \varepsilon$ $\quad \beta_2 = 20$

$\beta_3 := 22 \cdot \varepsilon$ $\quad \beta_3 = 27.5$

$class := if(\beta > \beta_1, if(\beta > \beta_2, if(\beta > \beta_3, 4, 3), 2), 1)$ $\quad class = 3$




6.1.5 **Local buckling**

6.1.5.(2) $\rho_c := if\left[\frac{\beta}{\varepsilon} \le 22, 1.0, \frac{32}{\left(\frac{\beta}{\varepsilon}\right)} - \frac{220}{\left(\frac{\beta}{\varepsilon}\right)^2}\right]$ $\quad \rho_c = 1$

$BC :=$ "A"

$t_{eff} := if(class \ge 4, t \cdot \rho_c, t)$ $\quad t_{eff} = 3.00 mm$

$A_{eff} := 4 \cdot (b - t) \cdot t_{eff}$ $\quad A_{eff} = 804 mm^2$

$I := \frac{b^4}{12} - \frac{(b - 2 \cdot t)^4}{12}$ $\quad I = 6.027 \times 10^5 mm^4$

$r := \sqrt{\frac{I}{A_{eff}}}$ $\quad r = 27.4 mm$

> According to 6.3.1.2 (1)
>
> $\lambda = \sqrt{\frac{A_{eff} \cdot f_o}{N_{cr}}}$
>
> Substitute
>
> $N_{cr} = \frac{\pi^2 \cdot E \cdot I}{l^2}$ $\quad r = \sqrt{\frac{I}{A_{eff}}}$
>
> Result
>
> $\lambda = \frac{l}{r} \cdot \frac{1}{\pi} \cdot \sqrt{\frac{f_o}{E}}$
>
> Note: $\lambda$ with a bar cannot be written in Mathcad

6.3.1 **Flexural buckling**

Table 6.8 $\quad k := 1 \quad l := k \cdot L$

Table 6.6 $\quad \alpha := 0.2 \quad \lambda_o := 0.1 \quad \kappa := 1$

See to the right $\quad \lambda := \frac{l}{r} \cdot \frac{1}{\pi} \cdot \sqrt{\frac{f_o}{E}}$ $\quad \lambda = 1.39$

(6.50) $\quad \phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - \lambda_o) + \lambda^2\right]$ $\quad \chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}}$

$\phi = 1.594 \quad \chi = 0.421$

**Axial force resistance allowing for local and flexural buckling**

(6.49) $\quad N_{b.Rd} := \kappa \cdot \chi \cdot \frac{f_o}{\gamma_{M1}} \cdot A_{eff}$ $\quad N_{b.Rd} = 49.2 kN$

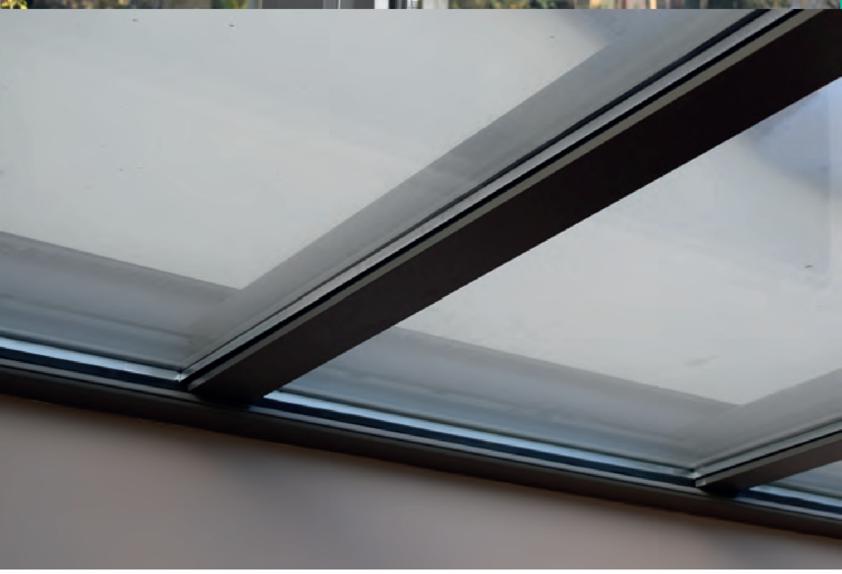
T Höglund aluMATTER 28.01.2021

## 5. SONUÇ

Yukarıda hesapları verilen raporda; Yavuz Metal Alüminyum A.Ş tarafından tasarlanan, plan ölçüleri 3,50 m x 3.00 m, yüksekliği ise 2,50 metreden başlayıp 3,00 metreye kadar çıkan alüminyum kış bahçesinin, düşey yükler altında hesapları yapılmıştır.

Kış bahçesinin yapılacağı bölgenin konumuna ve depremselliğine göre yatay yükler altında da tahkiki yapılmalıdır.

Yapılan hesaplar neticesinde kış bahçesini oluşturan profil kesitlerinin düşey yükler altında yeterli dayanıma ve kararlılığa sahip olduğu görülmüştür.



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