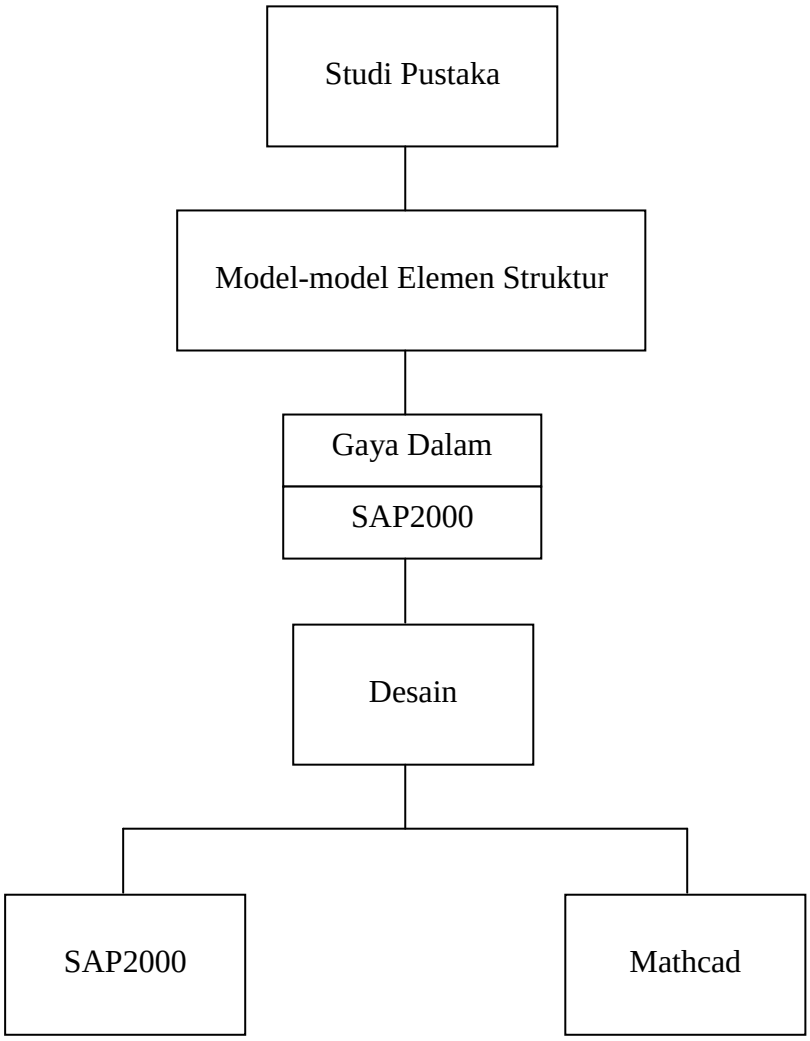
BAB III
METODE ANALISIS

3.1 Penyajian Laporan

Dalam penyajian bab ini dibuat kerangka agar memudahkan dalam pengerjaan laporan. Berikut ini adalah diagram alir tersebut :



Gambar III.1 Diagram Alir Perencanaan Penyajian Laporan

3.2 Model-model Elemen Struktur

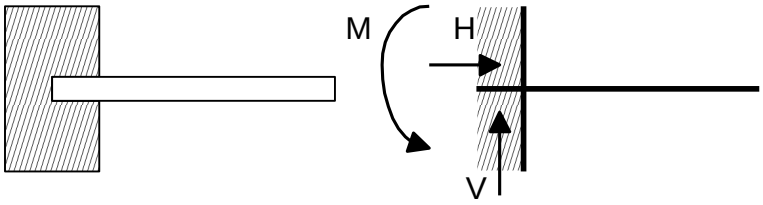
3.2.1 Balok Sederhana

Suatu balok yang disangga secara bebas pada kedua ujungnya disebut balok sederhana. Istilah "disangga secara bebas" menyatakan secara tidak langsung bahwa ujung penyangga hanya mampu menahan gaya-gaya pada batang dan tidak mampu menghasilkan momen. Dengan demikian tidak ada tahanan terhadap rotasi pada ujung batang jika batang mengalami tekukan karena pembebanan.

3.2.1.1 Jepit

Tumpuan jepit dapat memberikan reaksi vertikal, horisontal dan momen. Sifat-sifat tumpuan jepit :

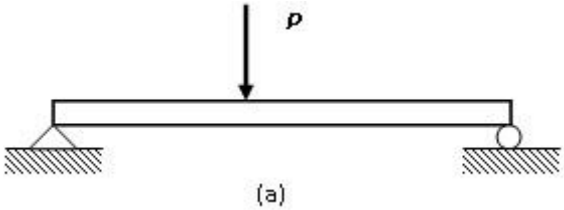
- Tidak dapat bergeser (vertikal maupun horisontal) dan berputar
- Dapat menahan gaya horisontal, gaya vertikal dan momen.

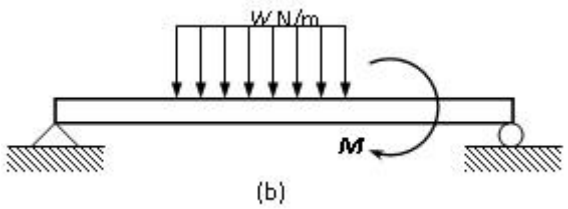


Gambar III.2 Tumpuan Jepit dan Reaksi yang Dapat Ditinjau

3.2.1.2 Sendi – Rol

Tumpuan sendi dapat memberikan reaksi vertikal dan horisontal. Sedangkan tumpuan rol hanya dapat memberikan reaksi vertikal.





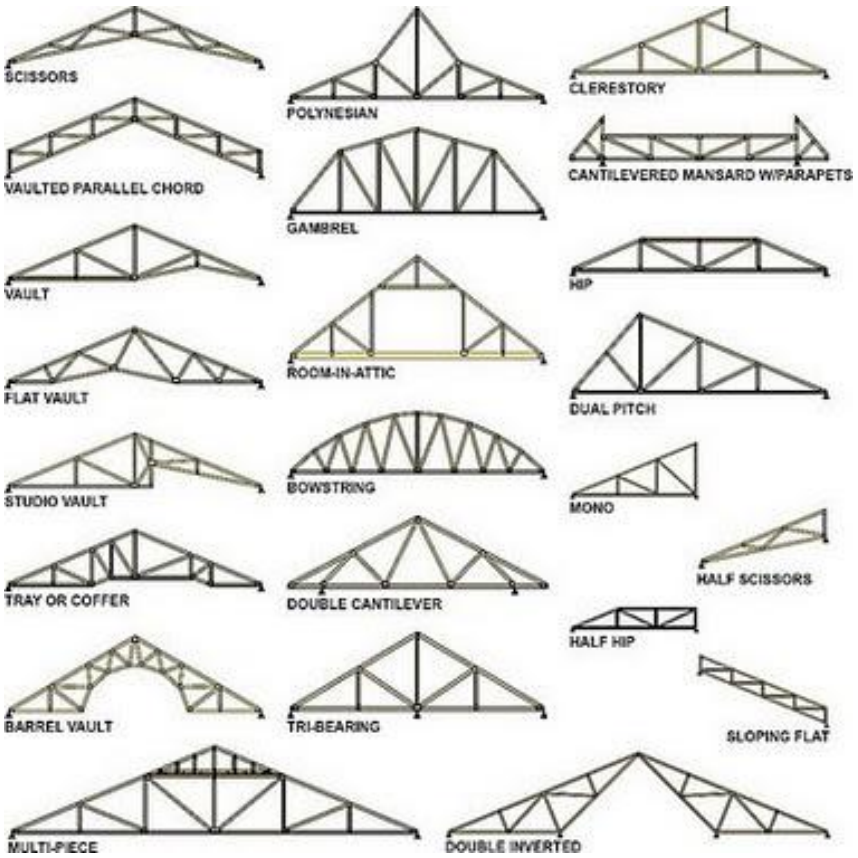
Gambar III.3 Ilustrasi Balok Sederhana (Sendi – Rol)

Perlu diperhatikan bahwa sedikitnya satu dari penyangga harus mampu menahan pergerakan horisontal sedemikian sehingga tidak ada gaya yang muncul pada arah sumbu balok.

Balok pada gambar 3.2(a) dikatakan dikenai gaya terkonsentrasi atau gaya tunggal, sedang batang pada gambar 3.2 (b) dibebani pasangan beban terdistribusi seragam.

3.2.2 Rangka Batang

Rangka batang adalah susunan elemen-elemen linier yang membentuk segitiga atau kombinasi segitiga dimana ujung-ujungnya dihubungkan pada satu titik dengan hubungan sendi, dan direncanakan untuk menerima beban yang cukup besar (dibandingkan berat sendirinya) yang bekerja pada titik-titik hubungnya.



Gambar III.4 Bentuk Rangka Batang Pada Umumnya
Sumber : http://baktidikara.blogspot.com/2011/06/baja-ringan-sebagai-konstruksi-penutup.html

3.2.2.1 Batang Tekan

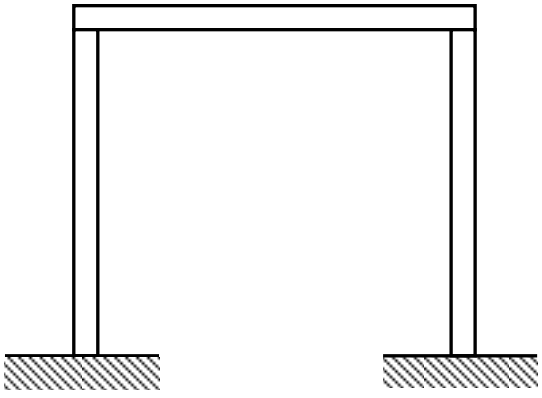
Akibat gaya aksial tekan, batang akan mengalami kemungkinan tiga perilaku tekuk, yaitu tekuk arah sumbu x, tekuk arah sumbu y, dan tekuk torsi. Prinsip desain terhadap batang tekan adalah mencari kapasitas maksimal penampang untuk menahan gaya aksial akibat beban luar yang dapat diterima batang sehingga perilaku tekuk yang terjadi masih dalam batas keamanan. Jika batang tidak kuat terhadap salah salah satu tekuk, makam dapat dilakukan perkuatan pada batang tersebut, namun efektifitas dan efisiensi penggunaan perkuatan harus tetap diperhatikan. Sehingga struktur tidak menjadi boros dan mudah dari segi pelaksanaannya.

3.2.2.2 Batang Tarik

Prinsip desain batang tarik dipengaruhi oleh dua hal yang harus dipenuhi agar struktur menjadi aman dan nyaman, yaitu *safety* dan *serviceability.* Secara prinsip *safety*, kekuatan sangat dipengaruhi oleh parameter luas penampang, luas penampang dalam hal ini adalah luas netto, yaitu luasan penampang dikurangi dengan luasan perlemahan akibat adanya baut. Secara prinsip *serviceability*, kelangsingan batang dapat menyebabkan lendutan meskipun secara struktural batang tersebut aman.

3.2.3 Kolom Portal

Kolom portal harus dibuat terus menerus dan lantai bawah sampai lantai atas, artinya letak kolom-kolom portal tidak boleh digeser pada tiap lantai, karena hal ini akan menghilangkan sifat kekakuan dari struktur rangka portalnya. Jadi harus dihindarkan denah kolom portal yang tidak sama untuk tiap-tiap lapis lantai. Ukuran kolom makin ke atas boleh makin kecil, sesuai dengan beban bangunan yang didukungnya makin ke atas juga makin kecil. Perubahan dimensi kolom harus dilakukan pada lapis lantai, agar pada suatu lajur kolom mempunyai kekakuan yang sama.



Gambar III.5 Contoh Bentuk Kolom Portal

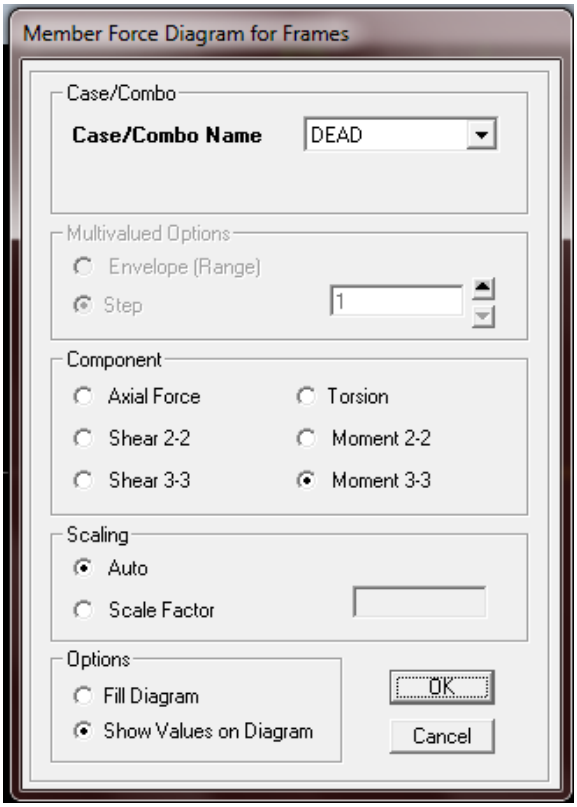
3.3 Gaya Dalam (Internal)

Gaya eksternal yang bekerja pada struktur akan menyebabkan timbulnya gaya internal di dalam elemen-elemen struktur. Gaya internal adalah gaya yang berasal dari dalam bangunan seperti beban bangunan itu sendiri. Beban yang ada pada bangunan terbagi dua yaitu beban mati dan beban hidup.

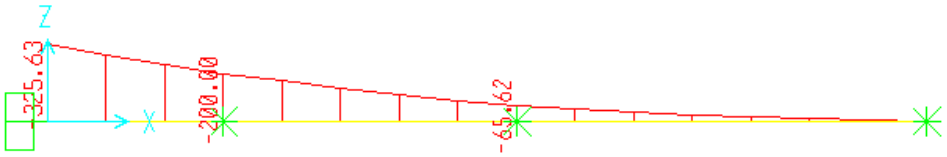
- Beban hidup : berat manusia, lemari, dan benda benda yang dapat dipindahkan.
- Beban mati : berat pondasi, kolom, dinding, dan sebagainya.

Gaya internal di dalam elemen yang paling umum terjadi adalah berupa tarik, tekan, lentur, geser, torsi, dan tumpu. Yang berkaitan dengan gaya-gaya internal adalah timbulnya tegangan dan regangan internal. Tegangan (*stress*) adalah ukuran intensitas gaya per satuan luas, dimana satuannya adalah lb/in^2 atau N/mm^2 (MPa), dan regangan (*strain*) adalah ukuran deformasi, dimana satuannya adalah in./in. atau mm/mm.

Berikut ini adalah hasil perhitungan gaya dalam menggunakan SAP2000. Langkah untuk memunculkan diagram bending momen adalah dengan menu perintah **Display – Show Forces/Stresses – Frame/Cables** kemudian akan ditampilkan menu **Member Force Diagram for Frames** dan pilih **Moment 3-3**.



Gambar III.6 Menu untuk Menentukan Diagram Bending Momen



Gambar III.7 Bending Momen Diagram (kN.m)

Perhitungan bending momen secara manual menggunakan Mathcad

$P_1 := 20kN$ $q := 5\frac{kN}{m}$ $L_1 := 1.5m$

$P_2 := 20kN$ $L_2 := 4m$

$P_3 := 10kN$ $L_3 := 7.5m$

$V_A := P_1 + P_2 + P_3 + (q \cdot L_3)$

$V_A = 87.5 \cdot kN$

$M_A := -(P_1 \cdot L_1) - (P_2 \cdot L_2) - (P_3 \cdot L_3) - [q \cdot L_3 \cdot (0.5 L_3)]$

$M_A = -325.625\,kN \cdot m$

$H_A := 0$

Segmen I

$N_1 := -H_A$ $\quad x_{1.1} := 0m$

$N_1 = 0 \cdot kN$

$Q_1 := V_A - q\,x_{1.1}$

$Q_1 = 87.5 \cdot kN$

$M_1 := M_A + V_A \cdot x_{1.1} - [q \cdot x_{1.1} \cdot (0.5 \cdot x_{1.1})]$

$M_1 = -325.625\,kN \cdot m$

$N_1 := -H_A$ $\quad x_{1.2} := 1m$

$N_1 = 0 \cdot kN$

$Q_1 := V_A - q\,x_{1.1}$

$Q_1 = 87.5 \cdot kN$

$M_1 := M_A + V_A \cdot x_{1.2} - [q \cdot x_{1.2} \cdot (0.5 \cdot x_{1.2})]$

$M_1 = -240.625\,kN \cdot m$

$N_1 := -H_A$ $\quad x_{1.3} := 1.5m$

$N_1 = 0 \cdot kN$

$Q_1 := V_A - q\,x_{1.3}$

$Q_1 = 80 \cdot kN$

$M_1 := M_A + V_A \cdot x_{1.3} - [q \cdot x_{1.3} \cdot (0.5 \cdot x_{1.3})]$

$M_1 = -200 \cdot kN \cdot m$

Segmen II

$N_2 := -H_A$ $x_{2.1} := 1.5m$

$N_2 = 0 \cdot kN$

$Q_2 := V_A - P_1 - q \cdot x_{2.1}$

$Q_2 = 60 \cdot kN$

$M_2 := M_A + V_A \cdot x_{2.1} - \left[P_1 \cdot \left(x_{2.1} - L_1\right)\right] - \left[q \cdot x_{2.1} \cdot \left(0.5 \cdot x_{2.1}\right)\right]$

$M_2 = -200 \cdot kN \cdot m$

$N_2 := -H_A$ $x_{2.2} := 2m$

$N_2 = 0 \cdot kN$

$Q_2 := V_A - P_1 - q \cdot x_{2.2}$

$Q_2 = 57.5 \cdot kN$

$M_2 := M_A + V_A \cdot x_{2.2} - \left[P_1 \cdot \left(x_{2.2} - L_1\right)\right] - \left[q \cdot x_{2.2} \cdot \left(0.5 \cdot x_{2.2}\right)\right]$

$M_2 = -170.625 kN \cdot m$

$N_2 := -H_A$ $x_{2.3} := 3m$

$N_2 = 0 \cdot kN$

$Q_2 := V_A - P_1 - q \cdot x_{2.3}$

$Q_2 = 52.5 \cdot kN$

$M_2 := M_A + V_A \cdot x_{2.3} - \left[P_1 \cdot \left(x_{2.3} - L_1\right)\right] - \left[q \cdot x_{2.3} \cdot \left(0.5 \cdot x_{2.3}\right)\right]$

$M_2 = -115.625 kN \cdot m$

$N_2 := -H_A$ $x_{2.4} := 4m$

$N_2 = 0 \cdot kN$

$Q_2 := V_A - P_1 - q \cdot x_{2.4}$

$Q_2 = 47.5 \cdot kN$

$M_2 := M_A + V_A \cdot x_{2.4} - \left[P_1 \cdot \left(x_{2.4} - L_1\right)\right] - \left[q \cdot x_{2.4} \cdot \left(0.5 \cdot x_{2.4}\right)\right]$

$M_2 = -65.625 kN \cdot m$

Segmen III

$N_3 := 0$ $x_{3.1} := 0m$

$N_3 = 0 \cdot kN$

$Q_3 := P_3 + q\,x_{3.1}$

$Q_3 = 10 \cdot kN$

$M_3 := -\left[q \cdot x_{3.1} \cdot \left(0.5 \cdot x_{3.1}\right)\right] - P_3 \cdot x_{3.1}$

$M_3 = 0 \cdot kN \cdot m$

$x_{3.2} := 1m$

$N_3 := 0$

$N_3 = 0 \cdot kN$

$Q_3 := P_3 + q\,x_{3.2}$

$Q_3 = 15 \cdot kN$

$M_3 := -\left[\left(q \cdot x_{3.2}\right) \cdot \left(0.5 \cdot x_{3.2}\right)\right] - P_3 \cdot x_{3.2}$

$M_3 = -12.5 \cdot kN \cdot m$

$N_3 := 0$ $x_{3.3} := 2m$

$N_3 = 0 \cdot kN$

$Q_3 := P_3 + q\,x_{3.3}$

$Q_3 = 20 \cdot kN$

$M_3 := -\left[q \cdot x_{3.3} \cdot \left(0.5 \cdot x_{3.3}\right)\right] - P_3 \cdot x_{3.3}$

$M_3 = -30 \cdot kN \cdot m$

$N_3 := 0$ $x_{3.5} := 3.5m$

$N_3 = 0 \cdot kN$

$Q_3 := P_3 + q\,x_{3.5}$

$Q_3 = 27.5 \cdot kN$

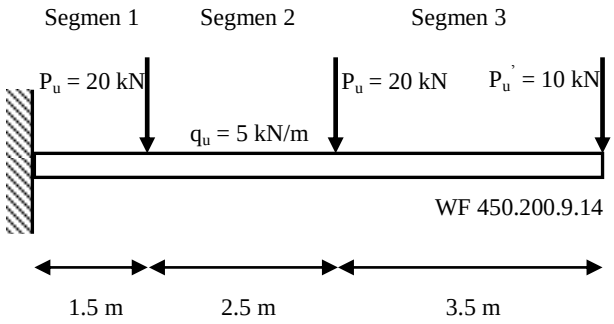
$M_3 := -\left[q \cdot x_{3.5} \cdot \left(0.5 \cdot x_{3.5}\right)\right] - P_3 \cdot x_{3.5}$

$M_3 = -65.625 \cdot kN \cdot m$

3.4 Desain

Ketika dihadapkan dalam desain sebuah struktur, kita perlu mempertimbangkan efisiensi struktur. Efisiensi yang dimaksud adalah desain struktur cukup kuat untuk menahan beban yang diberikan namun tetap hemat dalam hal biaya. Biaya biasanya berhubungan dengan banyak faktor, mulai dari jenis material, bentuk desain struktur, metode kerja, sumber daya, hingga biaya-biaya lainnya. Bentuk desain struktur merupakan bagian yang bisa dioptimasi untuk mencapai struktur yang efisien.

Berikut ini adalah contoh elemen struktur yang perhitungannya akan dibahas secara manual dan juga software :



Gambar III.8 Balok Kantilever yang Dibahas

Perhitungan secara manual sebuah struktur balok kantilever seperti gambar di atas akan dihitung dengan menggunakan Mathcad v.14.0.0.163.

- Properti penampang

E = 200000 MPa	I_x = 335000000 mm^4
G = 80000 Mpa	I_y =18700000 mm^4
BJ 37 : f_y = 240 MPa	S_x = 1488889 mm^3
WF 450.200	Z_x = 1621490 mm^3
A = 9680 mm^2	Z_y = 288500 mm^3
	J = 471815 mm^4
	C_w = 888698.8 mm^6

- Properti penampang terhadap pertambatan lateral

$r_y := \sqrt{\frac{I_y}{A}}$ $\quad r_y = 43.952\,mm$

$L_p := 1.76 r_y \cdot \sqrt{\frac{E}{f_y}}$ $\quad L_p = 2233\,mm$

$f_L := f_y - f_r$ $\quad f_L = 240\cdot MPa$

$X_1 := \frac{\pi}{S_x}\cdot\sqrt{\frac{E\cdot G\cdot J\cdot A}{2}}$ $\quad X_1 = 12754.28MPa$

$X_2 := 4\cdot\frac{C_w}{I_y}\left(\frac{S_x}{G\cdot J}\right)^2$ $\quad X_2 = 0\,\frac{1}{Pa^2}$

$L_r := r_y\cdot\left(\frac{X_1}{f_L}\right)\cdot\sqrt{1+\sqrt{1+X_2\cdot f_L^{\,2}}}$ $\quad L_r = 3303\,mm$

- Kuat nominal penampang

$M_r := S_x\cdot(f_y - f_r)$ $\quad M_r = 3.573\times 10^8\cdot N\cdot mm$

$M_{px} := Z_x\cdot f_y$ $\quad M_{px} = 3.892\times 10^8\cdot N\cdot mm$

Pertambahan lateral dipasang pada bagian yang ada beban terpusatnya

Segmen 1 $M_u := 325.625kN\cdot m$ $\quad \phi := 0.9$

$L_b := 1500mm$ $\quad L_p := 2233mm$

$C_b := 1$

$M_{cr} := C_b\cdot\frac{\pi}{L_b}\cdot\sqrt{E\cdot I_y\cdot G\cdot J+\left(\frac{\pi\cdot E}{L_b}\right)^2\cdot I_y\cdot C_w}$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px}-(M_{px}-M_r)\cdot\left(\frac{L_b-L_p}{L_r-L_p}\right)\right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 389.158kN\cdot m$ $\quad \phi\cdot M_{nx} = 350.242kN\cdot m$

$$Ratio := \begin{cases} \text{"OK"} & \text{if } \dfrac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases} \qquad \frac{M_u}{\phi \cdot M_{nx}} = 0.93$$

Ratio = "OK"

Segmen 2 $M_u := 200\,kN \cdot m$ $\phi := 0.9$

$L_b := 2500\,mm$ $L_p := 2233\,mm$

$C_b := 1$

$$M_{cr} := C_b \cdot \frac{\pi}{L_b} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_b}\right)^2 \cdot I_y \cdot C_w}$$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - \left(M_{px} - M_r\right) \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 381.218\,kN \cdot m$ $\phi \cdot M_{nx} = 343.097\,kN \cdot m$

$$Ratio := \begin{cases} \text{"OK"} & \text{if } \dfrac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases} \qquad \frac{M_u}{\phi \cdot M_{nx}} = 0.583$$

Ratio = "OK"

Segmen 3 $M_u := 65.625\,kN \cdot m$ $\phi := 0.9$

$L_b := 3500\,mm$ $L_p := 2233\,mm$ $L_r = 3.303\,m$

$$M_{cr} := C_b \cdot \frac{\pi}{L_b} \cdot \sqrt{\left(E \cdot I_y \cdot G \cdot J\right) + \left(\frac{\pi \cdot E}{L_b}\right)^2 \cdot I_y \cdot C_w}$$

$M_{cr} = 337.248\,kN \cdot m$ $M_{px} = 389.158\,kN \cdot m$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - \left(M_{px} - M_r\right) \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{cr} > M_p$ jadi $M_{nx} := M_{px}$

$M_{nx} = 389.158\,kN{\cdot}m$ $\phi \cdot M_{nx} = 350.242\,kN{\cdot}m$

$$Ratio := \begin{cases} \text{"OK"} & \text{if } \dfrac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases} \qquad \frac{M_u}{\phi \cdot M_{nx}} = 0.187$$

Ratio = "OK"

Balok kantilever tersebut juga dihitung menggunakan software SAP2000, sehingga menghasilkan perhitungan sebagai berikut :

Steel Stress Check Data AISC-LRFD99

File

AISC-LRFD99 STEEL SECTION CHECK
Combo : COMB1
Units : KN, m, C

Frame	: 3	Design Sect:	WF450-A		
X Mid	: 0.750	Design Type:	Beam		
Y Mid	: 0.000	Frame Type :	Ordinary Moment Frame		
Z Mid	: 0.000	Sect Class :	Compact		
Length	: 1.500	Major Axis :	0.000 degrees counterclockwise from local 3		
Loc	: 0.000	RLLF :	1.000		
Area	: 0.010	SMajor : 0.001	rMajor : 0.186	AVMajor:	0.004
IMajor	: 3.350E-04	SMinor : 1.870E-04	rMinor : 0.044	AVMinor:	0.005
IMinor	: 1.870E-05	ZMajor : 0.002	E : 200000000.00		
Ixy	: 0.000	ZMinor : 2.886E-04	Fy : 240000.000		

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
0.000	0.000	-325.625	0.000	-87.500	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(H1-1b)	0.930	=	0.000	+	0.930	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	0.000	1861.077	2090.880

MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	-325.625	350.242	1.000	1.000	1.000	1.000	1.000	1.186
Minor Moment	0.000	60.588	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	87.500	524.880	0.167	OK	0.000
Minor Shear	0.000	604.800	0.000	OK	0.000

Gambar III.9 Detail Perhitungan Check Tegangan pada Segmen 1

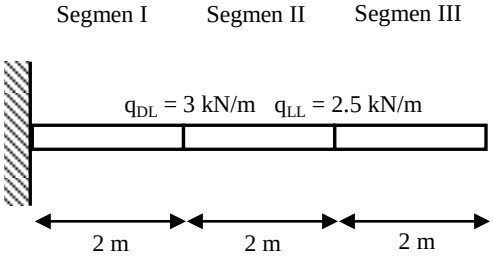
3.5. Contoh Perhitungan Model-model Elemen Struktur

Jepit

$q_{DL} := 3\frac{kN}{m}$ $\quad q_{LL} := 2.5\frac{kN}{m}$

$L := 6m$ $\quad q := (q_{DL}\cdot 1.2) + (q_{LL}\cdot 1.6)$

$q = 7.6\frac{kN}{m}$



E = 200000 MPa

G = 80000 Mpa

BJ 37 : f_y = 240 MPa

WF 450.200

A = 9680 mm^2

I_x = 335000000 mm^4

I_y =18700000 mm^4

S_x = 1488889 mm^3

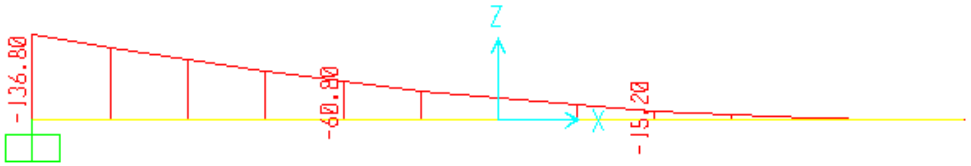
Z_x = 1621490 mm^3

Z_y = 288500 mm^3

J = 471815 mm^4

C_w = 888698.8 mm^6

Hasil bending momen (menggunakan SAP2000) :



Perhitungan bending momen secara manual (menggunakan Mathcad) :

$V_A := q\cdot L$

$V_A = 45.6\,kN$

$M_A := -V_A\cdot(0.5\cdot L)$

$M_A = -136.8\,kN\cdot m$

Segmen I

$M_1 := M_A + V_A\cdot x_{1.1} - \left[q\,x_{1.1}\cdot(0.5\cdot x_{1.1})\right]$ $\quad x_{1.1} := 0m$

$M_1 = -136.8\,kN\cdot m$

$M_1 := M_A + V_A \cdot x_{1.2} - \left[q\, x_{1.2} \cdot (0.5 \cdot x_{1.2})\right]$ $x_{1.2} := 1m$

$M_1 = -95 \cdot kN \cdot m$

$M_1 := M_A + V_A \cdot x_{1.3} - \left[q\, x_{1.3} \cdot (0.5 \cdot x_{1.3})\right]$ $x_{1.3} := 2m$

$M_1 = -60.8 \cdot kN \cdot m$

Segmen II

$M_2 := M_A + V_A \cdot x_{2.1} - \left[q\, x_{2.1} \cdot (0.5 \cdot x_{2.1})\right]$ $x_{2.1} := 2m$

$M_2 = -60.8 \cdot kN \cdot m$

$M_2 := M_A + V_A \cdot x_{2.2} - \left[q\, x_{2.2} \cdot (0.5 \cdot x_{2.2})\right]$ $x_{2.2} := 3m$

$M_2 = -34.2 \cdot kN \cdot m$

$M_2 := M_A + V_A \cdot x_{2.3} - \left[q\, x_{2.3} \cdot (0.5 \cdot x_{2.3})\right]$ $x_{2.3} := 4m$

$M_2 = -15.2 \cdot kN \cdot m$

Segmen III

$M_3 := -\left[q\, x_{3.1} \cdot (0.5 \cdot x_{3.1})\right]$ $x_{3.1} := 0m$

$M_3 = 0 \cdot kN \cdot m$

$M_3 := -\left[q\, x_{3.2} \cdot (0.5 \cdot x_{3.2})\right]$ $x_{3.2} := 1m$

$M_3 = -3.8 \cdot kN \cdot m$

$M_3 := -\left[q\, x_{3.3} \cdot (0.5 \cdot x_{3.3})\right]$ $x_{3.3} := 2m$

$M_3 = -15.2 \cdot kN \cdot m$

Mengitung ratio per segmen (menggunakan Mathcad) :

Properti penampang terhadap pertambatan lateral.

$r_y := \sqrt{\frac{I_y}{A}}$ $r_y = 43.952\,mm$

$L_p := 1.76 r_y \cdot \sqrt{\frac{E}{f_y}}$ $L_p = 2233\,mm$

$f_L := f_y - f_r$ $f_L = 240 \cdot MPa$

$X_1 := \frac{\pi}{S_x}\cdot\sqrt{\frac{E\cdot G\cdot J\cdot A}{2}}$ $\quad X_1 = 12754.28\,MPa$

$X_2 := 4\cdot\frac{C_w}{I_y}\left(\frac{S_x}{G\cdot J}\right)^2$ $\quad X_2 = 0\,\frac{1}{Pa^2}$

$L_r := r_y\cdot\left(\frac{X_1}{f_L}\right)\cdot\sqrt{1+\sqrt{1+X_2\cdot f_L^{\,2}}}$ $\quad L_r = 3303\,mm$

Kuat nominal penampang

$M_r := S_x\cdot(f_y - f_r)$ $\quad M_r = 3.573\times 10^8\cdot N\cdot mm$

$M_{px} := Z_x\cdot f_y$ $\quad M_{px} = 3.892\times 10^8\cdot N\cdot mm$

Segmen 1 $\quad M_u := 136.80\,kN\cdot m$ $\quad \phi := 0.9$

$L_b := 6000\,mm$ $\quad L_p := 2233\,mm$

$C_b := 1.14$

$M_{cr} := C_b\cdot\frac{\pi}{L_b}\cdot\sqrt{E\cdot I_y\cdot G\cdot J+\left(\frac{\pi\cdot E}{L_b}\right)^2\cdot I_y\cdot C_w}$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - (M_{px} - M_r)\cdot\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 224.27\,kN\cdot m$ $\quad \phi\cdot M_{nx} = 201.843\,kN\cdot m$

$$Ratio := \begin{cases} \text{"OK"} & \text{if } \frac{M_u}{\phi\cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$\frac{M_u}{\phi\cdot M_{nx}} = 0.678$

$Ratio = \text{"OK"}$

Segmen 2 $\quad M_u := 60.80\,kN\cdot m$ $\quad \phi := 0.9$

$L_b := 2000\,mm$ $\quad L_p := 2233\,mm$

$C_b := 1$

$M_{cr} := C_b\cdot\frac{\pi}{L_b}\cdot\sqrt{E\cdot I_y\cdot G\cdot J+\left(\frac{\pi\cdot E}{L_b}\right)^2\cdot I_y\cdot C_w}$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - \left(M_{px} - M_r\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 389.158\,kN \cdot m \qquad \phi \cdot M_{nx} = 350.242\,kN \cdot m$

$$Ratio := \begin{cases} \text{"OK"} & \text{if } \frac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\frac{M_u}{\phi \cdot M_{nx}} = 0.174$$

Ratio = "OK"

Segmen 3 $M_u := 15.20\,kN \cdot m \qquad \phi := 0.9$

$L_b := 2000\,mm \qquad L_p := 2233\,mm \qquad L_r = 3.303\,m$

$C_b := 1$

$$M_{cr} := C_b \cdot \frac{\pi}{L_b} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_b} \right)^2 \cdot I_y \cdot C_w}$$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - \left(M_{px} - M_r\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 389.158\,kN \cdot m \qquad \phi \cdot M_{nx} = 350.242\,kN \cdot m$

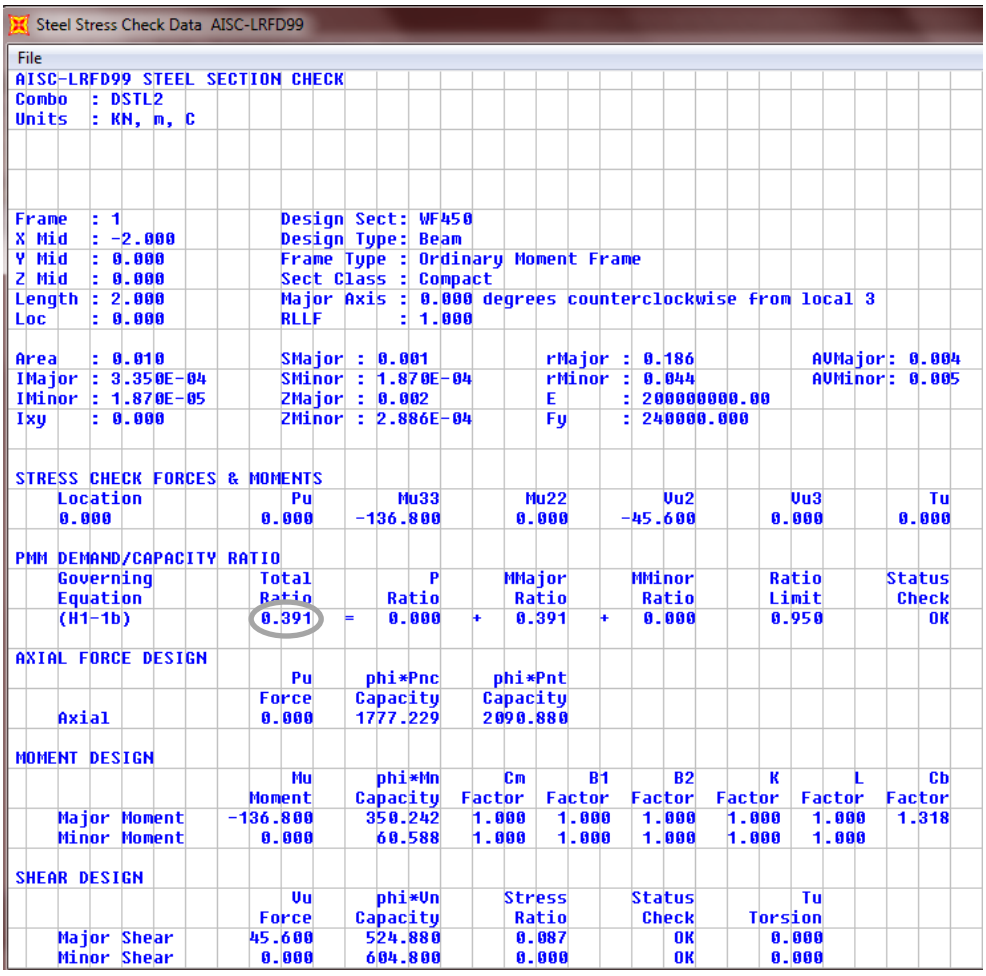
$$Ratio := \begin{cases} \text{"OK"} & \text{if } \frac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\frac{M_u}{\phi \cdot M_{nx}} = 0.043$$

Ratio = "OK"

Hasil ratio (menggunakan SAP2000) :

Segmen 1



Steel Stress Check Data AISC-LRFD99

File

AISC-LRFD99 STEEL SECTION CHECK
Combo : DSTL2
Units : KN, m, C

Frame : 1 — Design Sect: WF450
X Mid : -2.000 — Design Type: Beam
Y Mid : 0.000 — Frame Type : Ordinary Moment Frame
Z Mid : 0.000 — Sect Class : Compact
Length : 2.000 — Major Axis : 0.000 degrees counterclockwise from local 3
Loc : 0.000 — RLLF : 1.000

Area : 0.010 — SMajor : 0.001 — rMajor : 0.186 — AVMajor: 0.004
IMajor : 3.350E-04 — SMinor : 1.870E-04 — rMinor : 0.044 — AVMinor: 0.005
IMinor : 1.870E-05 — ZMajor : 0.002 — E : 200000000.00
Ixy : 0.000 — ZMinor : 2.886E-04 — Fy : 240000.000

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
0.000	0.000	-136.800	0.000	-45.600	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(H1-1b)	0.391	=	0.000	+	0.391	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	0.000	1777.229	2090.880

MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	-136.800	350.242	1.000	1.000	1.000	1.000	1.000	1.318
Minor Moment	0.000	60.588	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	45.600	524.880	0.087	OK	0.000
Minor Shear	0.000	604.800	0.000	OK	0.000

Segmen 2

Steel Stress Check Data AISC-LRFD99

File

AISC-LRFD99 STEEL SECTION CHECK
Combo : DSTL2
Units : KN, m, C

Frame : 2 Design Sect: WF450
X Mid : 0.000 Design Type: Beam
Y Mid : 0.000 Frame Type : Ordinary Moment Frame
Z Mid : 0.000 Sect Class : Compact
Length : 2.000 Major Axis : 0.000 degrees counterclockwise from local 3
Loc : 0.000 RLLF : 1.000

Area : 0.010 SMajor : 0.001 rMajor : 0.186 AVMajor: 0.004
IMajor : 3.350E-04 SMinor : 1.870E-04 rMinor : 0.044 AVMinor: 0.005
IMinor : 1.870E-05 ZMajor : 0.002 E : 200000000.00
Ixy : 0.000 ZMinor : 2.886E-04 Fy : 240000.000

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
0.000	0.000	-60.800	0.000	-30.400	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(H1-1b)	0.174	=	0.000	+	0.174	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	0.000	1777.229	2090.880

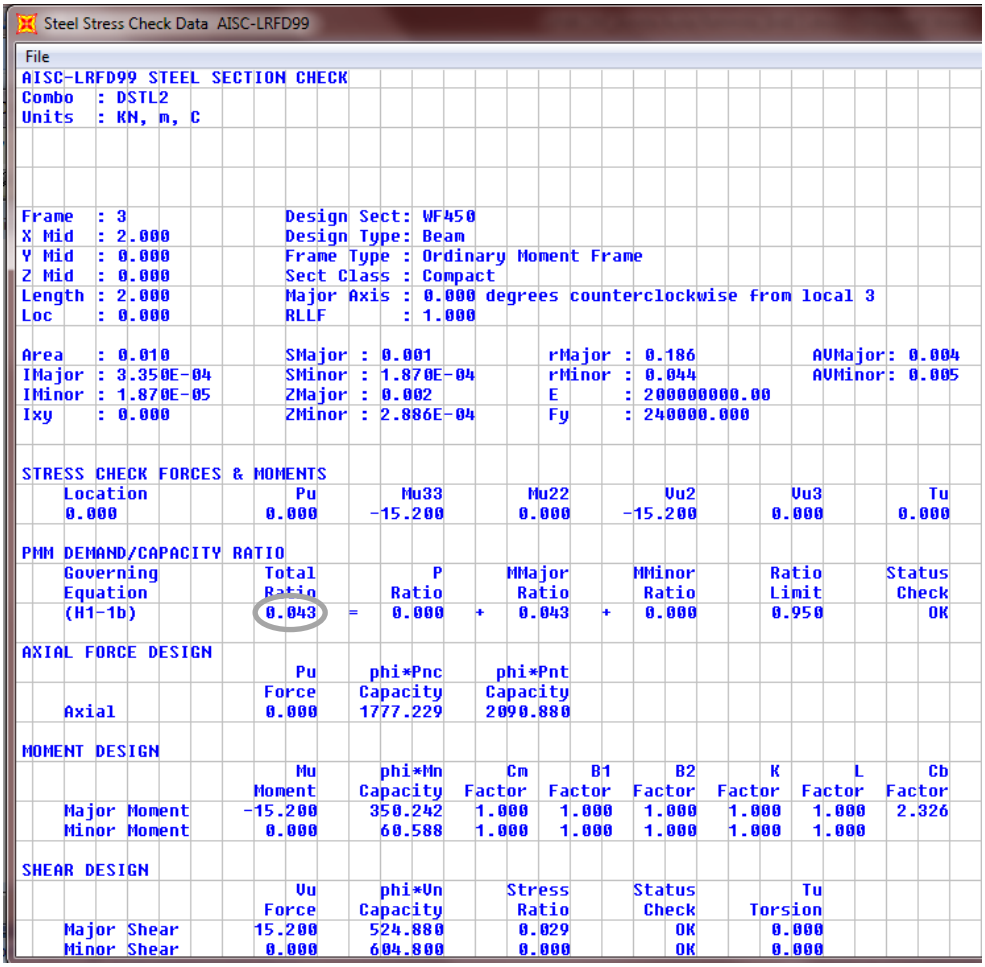
MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	-60.800	350.242	1.000	1.000	1.000	1.000	1.000	1.521
Minor Moment	0.000	60.588	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	30.400	524.880	0.058	OK	0.000
Minor Shear	0.000	604.800	0.000	OK	0.000

Segmen 3



Steel Stress Check Data AISC-LRFD99

File

AISC-LRFD99 STEEL SECTION CHECK
Combo : DSTL2
Units : KN, m, C

Frame : 3 Design Sect: WF450
X Mid : 2.000 Design Type: Beam
Y Mid : 0.000 Frame Type : Ordinary Moment Frame
Z Mid : 0.000 Sect Class : Compact
Length : 2.000 Major Axis : 0.000 degrees counterclockwise from local 3
Loc : 0.000 RLLF : 1.000

Area : 0.010 SMajor : 0.001 rMajor : 0.186 AVMajor: 0.004
IMajor : 3.350E-04 SMinor : 1.870E-04 rMinor : 0.044 AVMinor: 0.005
IMinor : 1.870E-05 ZMajor : 0.002 E : 200000000.00
Ixy : 0.000 ZMinor : 2.886E-04 Fy : 240000.000

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
0.000	0.000	-15.200	0.000	-15.200	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(H1-1b)	0.043	=	0.000	+	0.043	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	0.000	1777.229	2090.880

MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	-15.200	350.242	1.000	1.000	1.000	1.000	1.000	2.326
Minor Moment	0.000	60.588	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	15.200	524.880	0.029	OK	0.000
Minor Shear	0.000	604.800	0.000	OK	0.000

Sendi – Rol

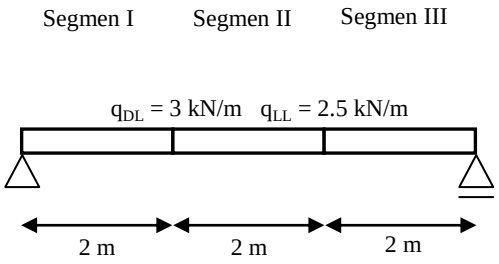
Jenis dan mutu baja yang dipakai dalam contoh perletakan sendi – rol sama dengan yang dipakai di contoh perletakan jepit.

$q_{DL} := 3\frac{kN}{m}$ $q_{LL} := 2.5\frac{kN}{m}$

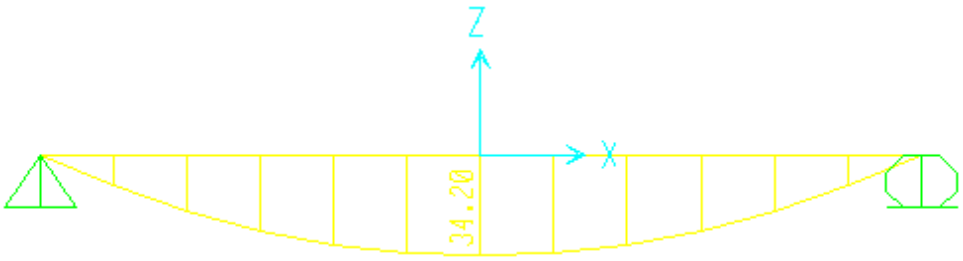
$L := 6m$

$q := (q_{DL} \cdot 1.2) + (q_{LL} \cdot 1.6)$

$q = 7.6\frac{kN}{m}$



Hasil bending momen (menggunakan SAP2000) :



Perhitungan bending momen secara manual (menggunakan Mathcad) :

$M_{max} := \left(\frac{1}{8}\right) \cdot q \cdot L^2$

$M_{max} = 34.2 \cdot kN \cdot m$

$V_A := \frac{[q \cdot L \cdot (0.5L)]}{L}$ $V_B := \frac{[q \cdot L \cdot (0.5L)]}{L}$

$V_A = 22.8 \cdot kN$ $V_B = 22.8 \cdot kN$

Segmen I

$M_1 := V_A \cdot x_{1.1} - q \cdot x_{1.1} \cdot (0.5 \cdot x_{1.1})$ $x_{1.1} := 0m$

$M_1 = 0 \cdot kN \cdot m$

$M_1 := V_A \cdot x_{1.2} - q \cdot x_{1.2} \cdot (0.5 \cdot x_{1.2})$ $x_{1.2} := 1m$

$M_{1.3} := V_A \cdot x_{1.3} - q \cdot x_{1.3} \cdot (0.5 \cdot x_{1.3})$ $x_{1.3} := 2m$

$M_1 = 30.4\,kN \cdot m$

Segmen II

$M_2 := V_A \cdot x_{2.1} - q \cdot x_{2.1} \cdot (0.5 \cdot x_{2.1})$ $x_{2.1} := 2m$

$M_2 = 30.4\,kN \cdot m$

$M_{2.2} := V_A \cdot x_{2.2} - q \cdot x_{2.2} \cdot (0.5 \cdot x_{2.2})$ $x_{2.2} := 3m$

$M_2 = 34.2\,kN \cdot m$

$M_{2.4} := V_A \cdot x_{2.4} - q \cdot x_{2.4} \cdot (0.5 \cdot x_{2.4})$ $x_{2.4} := 4m$

$M_2 = 30.4\,kN \cdot m$

Segmen III

$M_3 := V_B \cdot x_{3.1} - q \cdot x_{3.1} \cdot (0.5 \cdot x_{3.1})$ $x_{3.1} := 0m$

$M_3 = 0\,kN \cdot m$

$M_{3.2} := V_B \cdot x_{3.2} - q \cdot x_{3.2} \cdot (0.5 \cdot x_{3.2})$ $x_{3.2} := 1m$

$M_3 = 19\,kN \cdot m$

$M_{3.3} := V_B \cdot x_{3.3} - q \cdot x_{3.3} \cdot (0.5 \cdot x_{3.3})$ $x_{3.3} := 2m$

$M_3 = 30.4\,kN \cdot m$

Mengitung ratio (menggunakan Mathcad) :

Karena jenis dan mutu baja yang digunakan perletakan sendi – rol sama dengan yang digunakan pada perletakan jepit maka property penampang dan kuat nominal yang dihasilkan juga sama.

$M_u := 34.20kN \cdot m$ $\phi := 0.9$

$L_b := 6000mm$ $L_p = 1.674m$

$C_b := 1.14$

$$M_{cr} := C_b \cdot \frac{\pi}{L_b} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_b}\right)^2 \cdot I_y \cdot C_w}$$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - (M_{px} - M_r) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 70.832\,kN \cdot m \qquad \phi \cdot M_{nx} = 63.749\,kN \cdot m$

$$Ratio := \begin{cases} \text{"OK"} & \text{if } \frac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\frac{M_u}{\phi \cdot M_{nx}} = 0.536$$

$Ratio = \text{"OK"}$

Hasil ratio (menggunakan SAP2000) :

Steel Stress Check Data AISC-LRFD99

File

AISC-LRFD99 STEEL SECTION CHECK
Combo : DSTL2
Units : KN, m, C

Frame : 1 Design Sect: WF300
X Mid : 0.000 Design Type: Beam
Y Mid : 0.000 Frame Type : Ordinary Moment Frame
Z Mid : 0.000 Sect Class : Compact
Length : 6.000 Major Axis : 0.000 degrees counterclockwise from local 3
Loc : 3.000 RLLF : 1.000

Area : 0.005 SMajor : 4.807E-04 rMajor : 0.124 AVMajor: 0.002
IMajor : 7.210E-05 SMinor : 6.773E-05 rMinor : 0.033 AVMinor: 0.002
IMinor : 5.080E-06 ZMajor : 5.221E-04 E : 200000000.00
Ixy : 0.000 ZMinor : 1.042E-04 Fy : 240000.000

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
3.000	0.000	34.200	0.000	0.000	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(H1-1b)	0.538	=	0.000	+	0.538	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	0.000	207.639	1010.448

MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	34.200	63.546	1.000	1.000	1.000	1.000	1.000	1.136
Minor Moment	0.000	21.946	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	0.000	252.720	0.000	OK	0.000
Minor Shear	0.000	291.600	0.000	OK	0.000

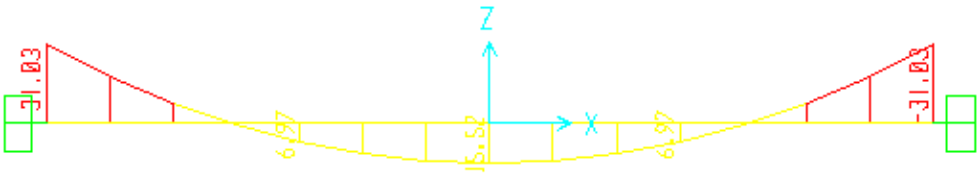
Kolom portal

Jenis dan mutu baja yang dipakai dalam contoh perletakan portal sama dengan yang dipakai di contoh perletakan jepit. Berikut ini akan dihitung momen dan ratio untuk baloknya saja.

$q_{DL} := 3\frac{kN}{m}$ $q_{LL} := 2.5\frac{kN}{m}$

$L := 7m$ $q := (q_{DL}\cdot 1.2) + (q_{LL}\cdot 1.6)$ $q = 7.6\frac{kN}{m}$

Hasil bending momen (menggunakan SAP2000) :



Perhitungan bending momen secara manual (menggunakan Mathcad) :

$V_A := \frac{[q\cdot L\cdot(0.5L)]}{L}$

$V_A = 26.6\,kN$

$V_B := V_A$

$M_A := -\left(\frac{1}{12}\right)\cdot q\cdot L^2$ $M_B := -\left(\frac{1}{12}\right)\cdot q\cdot L^2$

$M_A = -31.033\,kN\cdot m$ $M_B = -31.033\,kN\cdot m$

Segmen I

$M_1 := M_A + V_A\cdot x_{1.1} - q\cdot x_{1.1}\cdot(0.5\cdot x_{1.1})$ $x_{1.1} := 0m$

$M_1 = -31.033\,kN\cdot m$

$M_1 := M_A + V_A\cdot x_{1.2} - q\cdot x_{1.2}\cdot(0.5\cdot x_{1.2})$ $x_{1.2} := 1m$

$M_1 = -8.233\,kN\cdot m$

$M_{1.3} := M_A + V_A \cdot x_{1.3} - q \cdot x_{1.3} \cdot (0.5 \cdot x_{1.3})$ $x_{1.3} := 2m$

$M_1 = 6.967\,kN \cdot m$

Segmen II

$M_2 := M_A + V_A \cdot x_{2.1} - q \cdot x_{2.1} \cdot (0.5 \cdot x_{2.1})$ $x_{2.1} := 2m$

$M_2 = 6.967\,kN \cdot m$

$M_{2.2} := M_A + V_A \cdot x_{2.2} - q \cdot x_{2.2} \cdot (0.5 \cdot x_{2.2})$ $x_{2.2} := 3m$

$M_2 = 14.567\,kN \cdot m$

$M_{2.3} := M_A + V_A \cdot x_{2.3} - q \cdot x_{2.3} \cdot (0.5 \cdot x_{2.3})$ $x_{2.3} := 3.5m$

$M_2 = 15.517\,kN \cdot m$

$M_{2.4} := M_A + V_A \cdot x_{2.4} - q \cdot x_{2.4} \cdot (0.5 \cdot x_{2.4})$ $x_{2.4} := 4m$

$M_2 = 14.567\,kN \cdot m$

$M_{2.5} := M_A + V_A \cdot x_{2.5} - q \cdot x_{2.5} \cdot (0.5 \cdot x_{2.5})$ $x_{2.5} := 5m$

$M_2 = 6.967\,kN \cdot m$

Segmen III

$M_3 := M_B + V_B \cdot x_{3.1} - q \cdot x_{3.1} \cdot (0.5 \cdot x_{3.1})$ $x_{3.1} := 0m$

$M_3 = -31.033\,kN \cdot m$

$M_{3.2} := M_B + V_B \cdot x_{3.2} - q \cdot x_{3.2} \cdot (0.5 \cdot x_{3.2})$ $x_{3.2} := 1m$

$M_3 = -8.233\,kN \cdot m$

$M_{3.3} := M_B + V_B \cdot x_{3.3} - q \cdot x_{3.3} \cdot (0.5 \cdot x_{3.3})$ $x_{3.3} := 2m$

$M_3 = 6.967\,kN \cdot m$

Mengitung ratio per segmen (menggunakan Mathcad) :

Karena jenis dan mutu baja yang digunakan kolom portal sama dengan yang digunakan pada perletakan jepit maka property penampang dan kuat nominal yang dihasilkan juga sama.

Segmen 1 $M_u := 31.03\,kN\cdot m$ $\phi := 0.9$

$L_b := 2000\,mm$ $L_p := 2233\,mm$

$C_b := 1$

$$M_{cr} := C_b \cdot \frac{\pi}{L_b} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_b}\right)^2 \cdot I_y \cdot C_w}$$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - (M_{px} - M_r) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 389.158\,kN\cdot m$ $\phi \cdot M_{nx} = 350.242\,kN\cdot m$

$$Ratio := \begin{cases} \text{"OK"} & \text{if } \frac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\frac{M_u}{\phi \cdot M_{nx}} = 0.089$$

$Ratio = \text{"OK"}$

Segmen 2 $M_u := 15.52\,kN\cdot m$ $\phi := 0.9$

$L_b := 3000\,mm$ $L_p := 2233\,mm$

$C_b := 1$

$$M_{cr} := C_b \cdot \frac{\pi}{L_b} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_b}\right)^2 \cdot I_y \cdot C_w}$$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - (M_{px} - M_r) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 366.351\,kN\cdot m \qquad \phi \cdot M_{nx} = 329.716\,kN\cdot m$

$$Ratio := \begin{cases} \text{"OK"} & \text{if } \dfrac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\frac{M_u}{\phi \cdot M_{nx}} = 0.047$$

Ratio = "OK"

Segmen 3 $M_u := 31.03\,kN\cdot m \qquad \phi := 0.9$

$L_b := 2000\,mm \qquad L_p := 2233\,mm \qquad L_r = 3.303\,m$

$C_b := 1$

$$M_{cr} := C_b \cdot \frac{\pi}{L_b} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_b}\right)^2 \cdot I_y \cdot C_w}$$

$$M_{nx} := \begin{cases} M_{px} & \text{if } L_b \le L_p \\ \left[M_{px} - \left(M_{px} - M_r\right) \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] & \text{if } L_p \le L_b \le L_r \\ M_{cr} & \text{if } L_b > L_r \end{cases}$$

$M_{nx} = 389.158\,kN\cdot m \qquad \phi \cdot M_{nx} = 350.242\,kN\cdot m$

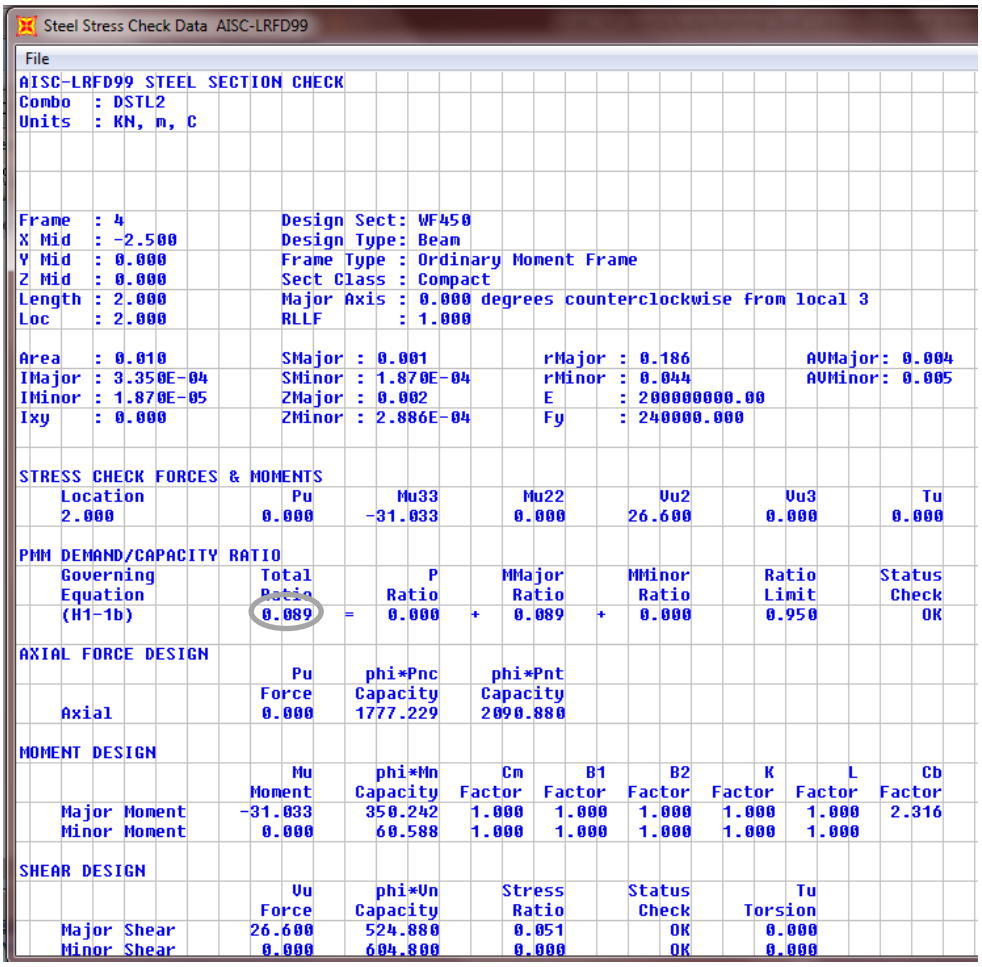
$$Ratio := \begin{cases} \text{"OK"} & \text{if } \dfrac{M_u}{\phi \cdot M_{nx}} < 1 \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\frac{M_u}{\phi \cdot M_{nx}} = 0.089$$

Ratio = "OK"

Hasil ratio (menggunakan SAP2000) :

Segmen 1



Steel Stress Check Data AISC-LRFD99

File

AISC-LRFD99 STEEL SECTION CHECK
Combo : DSTL2
Units : KN, m, C

Frame : 4 Design Sect: WF450
X Mid : -2.500 Design Type: Beam
Y Mid : 0.000 Frame Type : Ordinary Moment Frame
Z Mid : 0.000 Sect Class : Compact
Length : 2.000 Major Axis : 0.000 degrees counterclockwise from local 3
Loc : 2.000 RLLF : 1.000

Area : 0.010 SMajor : 0.001 rMajor : 0.186 AVMajor: 0.004
IMajor : 3.350E-04 SMinor : 1.870E-04 rMinor : 0.044 AVMinor: 0.005
IMinor : 1.870E-05 ZMajor : 0.002 E : 200000000.00
Ixy : 0.000 ZMinor : 2.886E-04 Fy : 240000.000

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
2.000	0.000	-31.033	0.000	26.600	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(H1-1b)	0.089	=	0.000	+	0.089	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	0.000	1777.229	2090.880

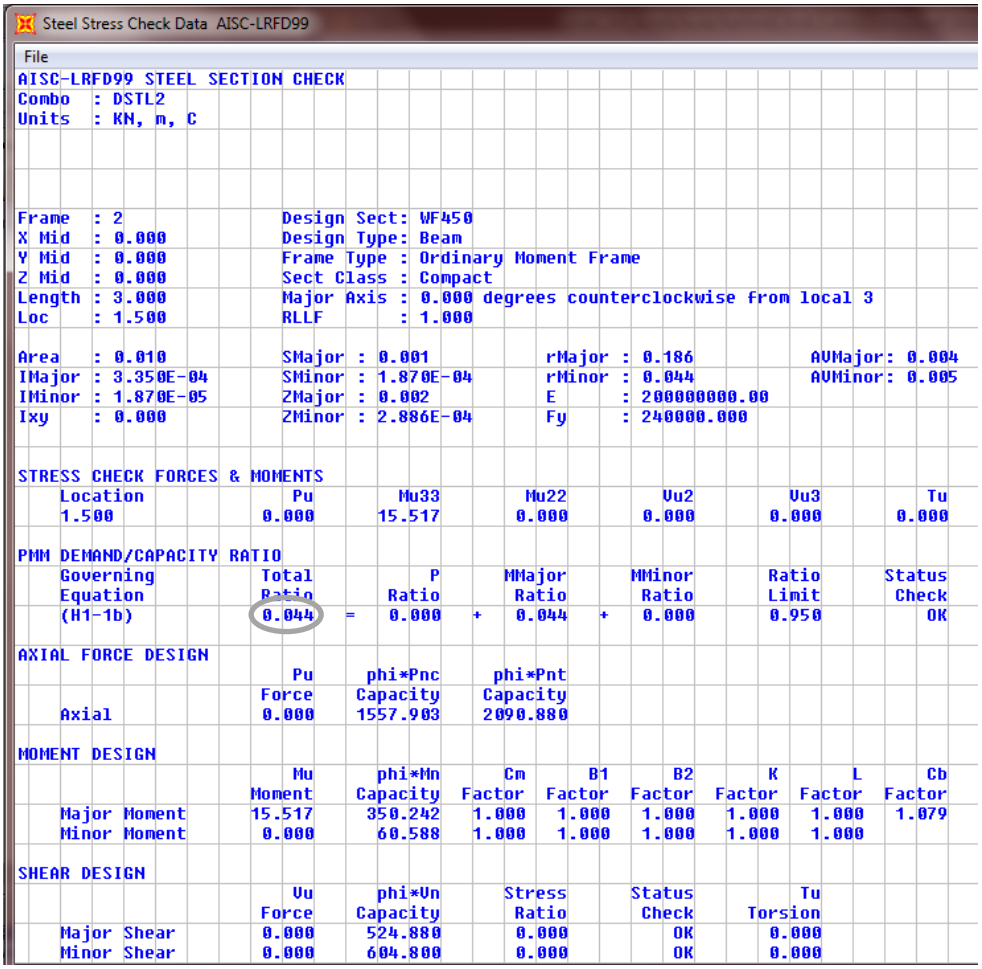
MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	-31.033	350.242	1.000	1.000	1.000	1.000	1.000	2.316
Minor Moment	0.000	60.588	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	26.600	524.880	0.051	OK	0.000
Minor Shear	0.000	604.800	0.000	OK	0.000

Segmen 2



Steel Stress Check Data AISC-LRFD99

File

AISC-LRFD99 STEEL SECTION CHECK
Combo : DSTL2
Units : KN, m, C

Frame : 2 Design Sect: WF450
X Mid : 0.000 Design Type: Beam
Y Mid : 0.000 Frame Type : Ordinary Moment Frame
Z Mid : 0.000 Sect Class : Compact
Length : 3.000 Major Axis : 0.000 degrees counterclockwise from local 3
Loc : 1.500 RLLF : 1.000

Area : 0.010 SMajor : 0.001 rMajor : 0.186 AVMajor: 0.004
IMajor : 3.350E-04 SMinor : 1.870E-04 rMinor : 0.044 AVMinor: 0.005
IMinor : 1.870E-05 ZMajor : 0.002 E : 200000000.00
Ixy : 0.000 ZMinor : 2.886E-04 Fy : 240000.000

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
1.500	0.000	15.517	0.000	0.000	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(H1-1b)	0.044	=	0.000	+	0.044	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	0.000	1557.903	2090.880

MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	15.517	350.242	1.000	1.000	1.000	1.000	1.000	1.079
Minor Moment	0.000	60.588	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	0.000	524.880	0.000	OK	0.000
Minor Shear	0.000	604.800	0.000	OK	0.000

Segmen 3

Steel Stress Check Data AISC-LRFD99

File

AISC-LRFD99 STEEL SECTION CHECK
Combo : DSTL2
Units : KN, m, C

Frame	: 5	Design Sect:	WF450		
X Mid	: 2.500	Design Type:	Beam		
Y Mid	: 0.000	Frame Type :	Ordinary Moment Frame		
Z Mid	: 0.000	Sect Class :	Compact		
Length	: 2.000	Major Axis :	0.000 degrees counterclockwise from local 3		
Loc	: 2.000	RLLF :	1.000		
Area	: 0.010	SMajor :	0.001	rMajor :	0.186
IMajor	: 3.350E-04	SMinor :	1.870E-04	rMinor :	0.044
IMinor	: 1.870E-05	ZMajor :	0.002	E :	200000000.00
Ixy	: 0.000	ZMinor :	2.886E-04	Fy :	240000.000

AVMajor: 0.004
AVMinor: 0.005

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
2.000	0.000	-31.033	0.000	26.600	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(H1-1b)	0.089	=	0.000	+	0.089	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	0.000	1777.229	2090.880

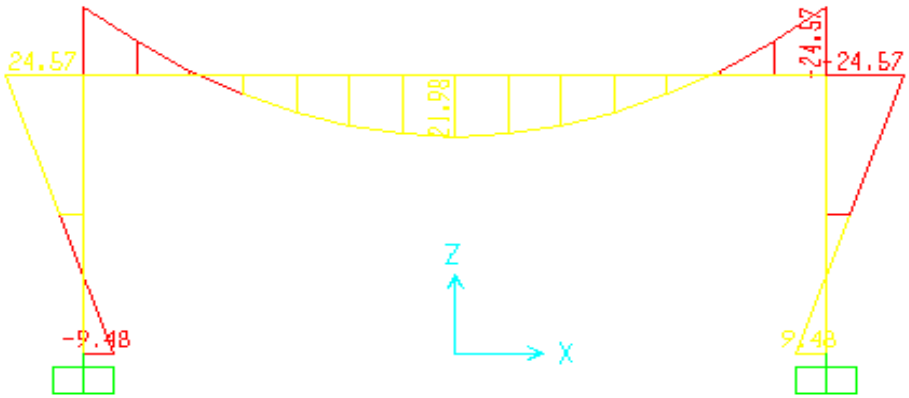
MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	-31.033	350.242	1.000	1.000	1.000	1.000	1.000	2.316
Minor Moment	0.000	60.588	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	26.600	524.880	0.051	OK	0.000
Minor Shear	0.000	604.800	0.000	OK	0.000

Menghitung momen pendekatan pada kolom portal dengan cara Slope Deflection. Berikut ini adalah hasil yang ditampilkan oleh SAP2000.



Dan ini adalah hasil perhitungan manual (menggunakan Mathcad) :

$M_{FAB} := \left(\frac{1}{12}\right) \cdot q \cdot L^2$ $M_{FBA} := -\left(\frac{1}{12}\right) \cdot q \cdot L^2$

$M_{FAB} = 31.033\,kN \cdot m$ $M_{FBA} = -31.033\,kN \cdot m$

$\theta_C := 0$ $M_{FAC} := 0$

$\theta_D := 0$ $M_{FBD} := 0$

$k_{AC} := \left(\frac{1}{1.5}\right) kN \cdot m$ $M_{FCA} := 0$

$k_{BD} := \left(\frac{1}{1.5}\right) kN \cdot m$ $M_{FDB} := 0$

$k_{AB} := \left(\frac{1}{1.75}\right) kN \cdot m$

$M_{AB} := M_{FAB} + k_{AB} \cdot (-2 \cdot \theta_A - \theta_B)$

$M_{BA} := M_{FBA} + k_{AB} \cdot (-2 \cdot \theta_B - \theta_A)$

$M_{AC} := M_{FAC} + k_{AC} \cdot (-2 \cdot \theta_A - \theta_C)$

$M_{CA} := M_{FCA} + k_{AC} \cdot (-2 \cdot \theta_C - \theta_A)$

$M_{BD} := M_{FBD} + k_{BD} \cdot (-2 \cdot \theta_B - \theta_D)$

$M_{DB} := M_{FDB} + k_{BD} \cdot (-2 \cdot \theta_D - \theta_B)$

Syarat batas :

$\Sigma M_A = 0 \rightarrow M_{AB} + M_{AC} = 0$

Hasilnya : $2.476\ \theta_A + 0.571\ \theta_B = 31.033$

$\Sigma M_B = 0 \rightarrow M_{BA} + M_{BD} = 0$

Hasilnya : $0.571\ \theta_A + 2.476\ \theta_B = 31.033$

Dengan cara eliminasi atau distribusi didapatkan :

$\theta_A = 16.292$ dan $\theta_B = -16.292$

Hasil momen yang didapat :

$M_{AB} = 21.723\,kN\cdot m$

$M_{BA} = -21.723\,kN\cdot m$

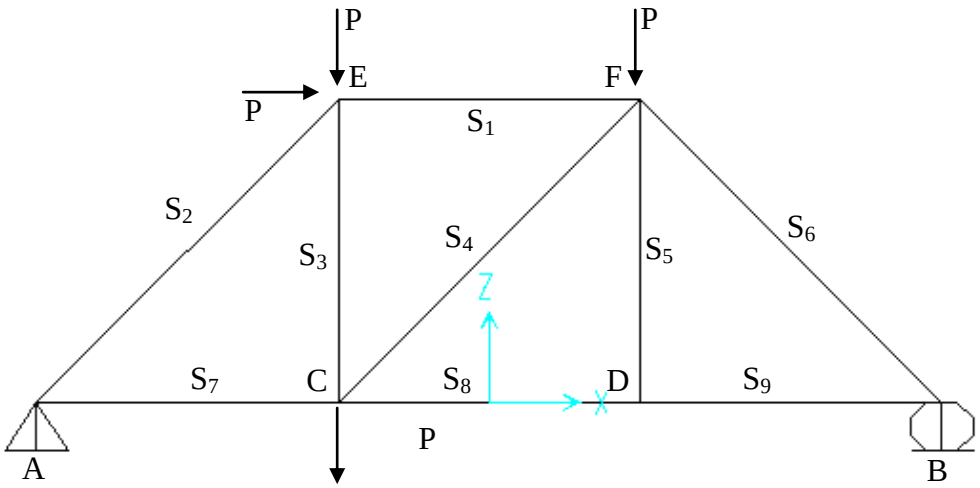
$M_{AC} = -21.723\,kN\cdot m$

$M_{CA} = -10.862\,kN\cdot m$

$M_{BD} = 21.723\,kN\cdot m$

$M_{DB} = 10.862\,kN\cdot m$

Rangka batang



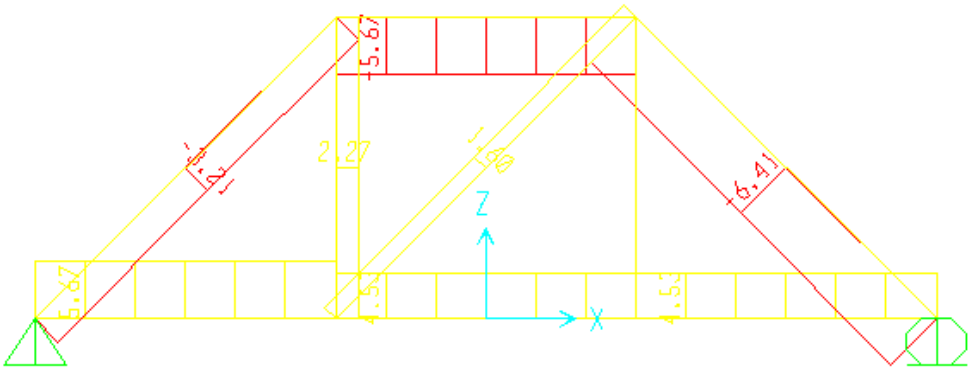
$q_{DL} := 1.5\frac{kN}{m}$ $q_{LL} := 1\frac{kN}{m}$ $L := 9m$

$q := (q_{DL}\cdot 1.2) + (q_{LL}\cdot 1.6)$ $L_1 := 3m$

$q = 3.4\frac{kN}{m}$ $h := 3m$

$P := 3.4kN$

Hasil keseimbangan titik (menggunakan SAP2000) :



Perhitungan keseimbangan titik secara manual (menggunakan Mathcad) :

$$V_A := \frac{\left[-P \cdot h + P \cdot L_1 + P \cdot \left(2 \cdot L_1\right)\right]}{L}$$

$V_A = 2.267\,kN$

$$V_B := \frac{\left[P \cdot h + P \cdot L_1 + P \cdot \left(2 \cdot L_1\right)\right]}{L}$$

$V_B = 4.533\,kN$

$H_A := -P$

$H_A = -3.4\,kN$

Joint A

$\Sigma Fy := 0$

$$S_2 := \frac{-V_A}{0.5\sqrt{2}}$$

$S_2 = -3.206\,kN$

$\Sigma Fx := 0$

$S_7 := P - S_2 \cdot \left(0.5\sqrt{2}\right)$

$S_7 = 5.667\,kN$

Joint E

$\Sigma Fy := 0$

$S_3 := -S_2 \cdot \left(0.5\sqrt{2}\right)$

$S_3 = 2.267\,kN$

$\Sigma Fx := 0$

$S_1 := -P + S_2 \cdot \left(0.5\sqrt{2}\right)$

$S_1 = -5.667\,kN$

Joint C

$\Sigma Fy := 0$

$S_4 := \frac{(P - S_3)}{0.5\sqrt{2}}$

$S_4 = 1.603\,kN$

$\Sigma Fx := 0$

$S_8 := S_7 - S_4 \cdot (0.5\sqrt{2})$

$S_8 = 4.533\,kN$

Joint D

$\Sigma Fy := 0$

$S_5 := 0$

$\Sigma Fx := 0$

$S_9 := S_8$

$S_9 = 4.533\,kN$

Joint F

$\Sigma Fy := 0$

$S_6 := \frac{\left[-P - S_5 - S_4 \cdot (0.5\sqrt{2})\right]}{0.5\sqrt{2}}$

$S_6 = -6.411\,kN$

$\Sigma Fy := 0$

$S_{6.1} := \frac{S_1 + S_4 \cdot (0.5\sqrt{2})}{0.5\sqrt{2}}$

$S_{6.1} = -6.411\,kN$

Joint B

$\Sigma Fy := 0$

$S_{6.2} := \frac{-V_B}{0.5\sqrt{2}}$

$S_{6.2} = -6.411\,kN$

$\Sigma Fx := 0$

$S_{9.1} := -S_6 \cdot (0.5\sqrt{2})$

$S_{9.1} = 4.533\,kN$

Kuat tarik batang profil siku sama kaki dan perhitungan blok geser :

Data Penampang Profil siku 120.120.8

Tinggi Penampang	$d := 120mm$	Luas Bruto Penampang	$A := 18.76cm^2$
Lebar Flens	$b := 120mm$	Tebal Profil	$t := 8mm$
Lokasi Titik Berat	$xbar := 3.24cm$		

Data Bahan Profil L dan Pelat Buhul menggunakan mutu baja BJ.37

$Fy := 240MPa$ $Fu := 370MPa$

Data Sambungan Diameter baut : $db := \frac{3}{4}in$ Banyak baut : $nbaut := 4$

Data Jarak dipasang berseling pada sisi panjang (150mm) dengan jarak seragam

$g1 := 45mm$ $s := 50mm$ $g := 45mm$ $d1 := d - g1 - g$ $d1 = 30\cdot mm$

Mencari Luas Netto Effektif

Diameter lubang (anggap lubang standar) $dhole := db + \frac{1}{8}in$ $dhole = 0.022\cdot m$

$L := (nbaut - 1)s$ $L = 0.15\,m$

$U := \min\left[\left(1 - \frac{xbar}{L}\right), 0.9\right]$ $U = 0.784$

Jalur lurus melalui 1 lubang baut: $An1 := A - (dhole)t$ $An1 = 16.982\cdot cm^2$

Jalur belok melalui 2 lubang baut berseling: $An2 := A - 2(dhole)t + \frac{s^2 t}{4g}$ $An2 = 16.315\cdot cm^2$

$Ae := U\cdot\min(An1, An2)$ $Ae = 12.791\cdot cm^2$

Fraktur pada penampang neto	$\phi t := 0.75$	$Pu1 := \phi t\cdot Fu\cdot Ae$	$Pu1 = 354.952\cdot kN$
Leleh pada penampang bruto	$\phi t := 0.9$	$Pu2 := \phi t\cdot Fy\cdot A$	$Pu2 = 405.216\cdot kN$

Geser blok (diperiksa blok yang lebarnya = d1=30mm)

$Agv := 5\cdot s\cdot t$	$Agv = 20\cdot cm^2$
$Agt := d1\cdot t$	$Agt = 2.4\cdot cm^2$
$Anv := Agv - 2.5\cdot dhole\cdot t$	$Anv = 15.555\cdot cm^2$
$Ant := Agt - 0.5\cdot dhole\cdot t$	$Ant = 1.511\cdot cm^2$

$\phi := 0.75$ $0.6Fu\cdot Anv = 345.321\cdot kN$ $Fu\cdot Ant = 55.907\cdot kN$

$$Rn := \begin{cases} [\phi\cdot(0.6\cdot Fu\cdot Anv + Fy\cdot Agt)] & \text{if } 0.6Fu\cdot Anv > Fu\cdot Ant \quad \text{(leleh tarik dan fraktur geser)} \\ [\phi\cdot(0.6\cdot Fy\cdot Agv + Fu\cdot Ant)] & \text{otherwise} \quad \text{(leleh geser dan fraktur tarik)} \end{cases}$$

$Rn = 302.191\cdot kN$

Kuat tarik batang profil siku adalah $Pn1 := \min(Pu1, Pu2, Rn)$ $Pn1 = 302.191\cdot kN$

Desain tekan profil siku sama kaki :

Panjang Efektif $kLx := 4.2426m$ $kLy := 4.2426m$ $kLz := 4.2426m$

Data Penampang Profil L120.120.8

$A := 18.76cm^2$ $b := 120mm$ $t := 8mm$

$imin := 2.38cm$ $imaks := 4.67cm$ Jarak titik berat $e := 3.24cm$

Data Bahan $E := 2 \cdot 10^5 MPa$ $G := 80000MPa$ $Fy := 240MPa$ (Bj. 37)

Beban Terfaktor $Pu := 6.41kN$

Cek kelangsingan penampang

$$\frac{b}{t} = 15 \qquad 0.45\sqrt{\frac{E}{Fy}} = 12.99$$

$$Cek := \begin{cases} \text{"Penampang Tidak Langsing"} & \text{if } \frac{b}{t} \le 0.45\sqrt{\frac{E}{Fy}} \\ \text{"Penampang LANGSING"} & \text{otherwise} \end{cases}$$

Cek = "Penampang LANGSING"

Menghitung kuat tekuk lentur untuk sumbu η (sumbu lemah)

$rx := imin$ $rx = 2.38 \cdot cm$

$$\frac{kLx}{rx} = 178.261 \qquad Cek := \begin{cases} \text{"OK"} & \text{if } \frac{kLx}{rx} \le 200 \\ \text{"Tidak OK!"} & \text{otherwise} \end{cases}$$

Cek = "OK"

$$\lambda c := \frac{kLx}{rx \cdot \pi}\sqrt{\frac{Fy}{E}} \qquad \lambda c = 1.97$$

$$Fcr1 := \begin{cases} \left(0.658^{\lambda c^2}\right) \cdot Fy & \text{if } \lambda c \le 1.5 \\ \left(\frac{0.877}{\lambda c^2} \cdot Fy\right) & \text{if } \lambda c > 1.5 \end{cases} \qquad Fcr1 = 54.48 \cdot MPa$$

Menghitung Kuat Tekuk Torsi Lentur untuk Sumbu ζ

Prosedur yang digunakan adalah AISC Appendix E3, bukan AISC Section E3, karena AISC Section E3 hanya untuk profil siku ganda atau profil T.

$Iy := imaks^2 \cdot A$ $Iy = 409.13 \cdot cm^4$

$Ix := imin^2 \cdot A$ $Ix = 106.26 \cdot cm^4$

Hati - hati, η dan ζ ini adalah sumbu miring 45 derajat, bukan sumbu sejajar kaki

$$J := \frac{2}{3}\left(b - \frac{t}{2}\right) \cdot t^3 \qquad J = 3.96 \cdot cm^4$$

$$Cw := \frac{2}{36}\left[(b - 0.5t)^3 \cdot t^3\right] \qquad Cw = 44.399 \cdot cm^6$$

$x0 := 0mm$ $y0 := \frac{(e - 0.5 \cdot t)}{\cos(45deg)}$ $y0 = 4.016 \cdot cm$

$r0 := \sqrt{x0^2 + y0^2 + \frac{(Ix + Iy)}{A}}$ $r0^2 = 43.605 \cdot cm^2$ $y0^2 = 16.131 \cdot cm^2$

(notasi AISC pakai tanda bar dan indeks nol)

$H := 1 - \left(\frac{x0^2 + y0^2}{r0^2}\right)$ $H = 0.63$

$ry := \sqrt{\frac{Iy}{A}}$ $ry = 4.67 \cdot cm$

$\lambda y := \frac{kLy}{ry}$ $\lambda y = 90.85$

$Fey := \frac{\pi^2 E}{\lambda y^2}$ $Fey = 239.17 \cdot MPa$

$Fez := \left[\frac{\pi^2 E\, Cw}{(kLz)^2} + G \cdot J\right] \cdot \frac{1}{A \cdot r0^2}$ $Fez = 387.82 \cdot MPa$

$Fe := \frac{Fey + Fez}{2 \cdot H}\left[1 - \sqrt{1 - \frac{4 \cdot Fey \cdot Fez \cdot H}{(Fey + Fez)^2}}\right]$ $Fe = 180.77 \cdot MPa$

$\lambda e := \sqrt{\frac{Fy}{Fe}}$ $\lambda e = 1.15$

$$Fcr2 := \begin{cases} \left(0.658^{\lambda e^2}\right) \cdot Fy & \text{if } \lambda e \le 1.5 \\ \left(\frac{0.877}{\lambda e^2} \cdot Fy\right) & \text{if } \lambda e > 1.5 \end{cases}$$

$Fcr2 = 137.68 \cdot MPa$

Kuat Desain Batang Tekan ini

$\phi c := 0.85$ $Fcr := \min(Fcr1, Fcr2)$ $Fcr = 54.48 \cdot MPa$

$\phi c \cdot Fcr \cdot A = 86.87 \cdot kN$

$$Cek := \begin{cases} \text{"OK"} & \text{if } \phi c \cdot Fcr \cdot A > Pu \\ \text{"Tidak OK"} & \text{otherwise} \end{cases}$$

$Cek = \text{"OK"}$

Menghitung ratio dari P (tarik dan tekan) yang dihasilkan dari perhitungan rangka batang :

S_7 = 5.677 kN (tarik)

Ratio = P_{S7} / 2 x P_{u1} = 0.007

S_6 = - 6.411 kN (tekan)

Ratio = P_{S7} / 2 x (ϕ_C x Fcr x A) = 0.037

Hasil ratio (menggunakan SAP2000) :

S_7 = 5.677 kN (tarik)

Steel Stress Check Data AISC-LRFD99

File

Units : KN, m, C

Frame	: 1	Design Sect: 120.120.8							
X Mid	: -3.000	Design Type: Beam							
Y Mid	: 0.000	Frame Type : Ordinary Moment Frame							
Z Mid	: 0.000	Sect Class : Slender							
Length	: 3.000	Major Axis : 0.000 degrees counterclockwise from local 3							
Loc	: 3.000	RLLF : 1.000							
Area	: 0.002	SMajor : 2.999E-05		rMajor : 0.038		AVMajor: 9.600E-04			
IMajor	: 2.610E-06	SMinor : 2.999E-05		rMinor : 0.038		AVMinor: 9.600E-04			
IMinor	: 2.610E-06	ZMajor : 5.401E-05		rMax : 0.047		E : 200000000.00			
Ixy	: -1.557E-06	ZMinor : 5.401E-05		rMin : 0.024		Fy : 240000.000			
Theta	: 45.000								

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
3.000	5.667	0.000	0.000	0.000	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(SAM 6-1b)	0.007	=	0.007	+	0.000	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	5.667	176.642	400.896

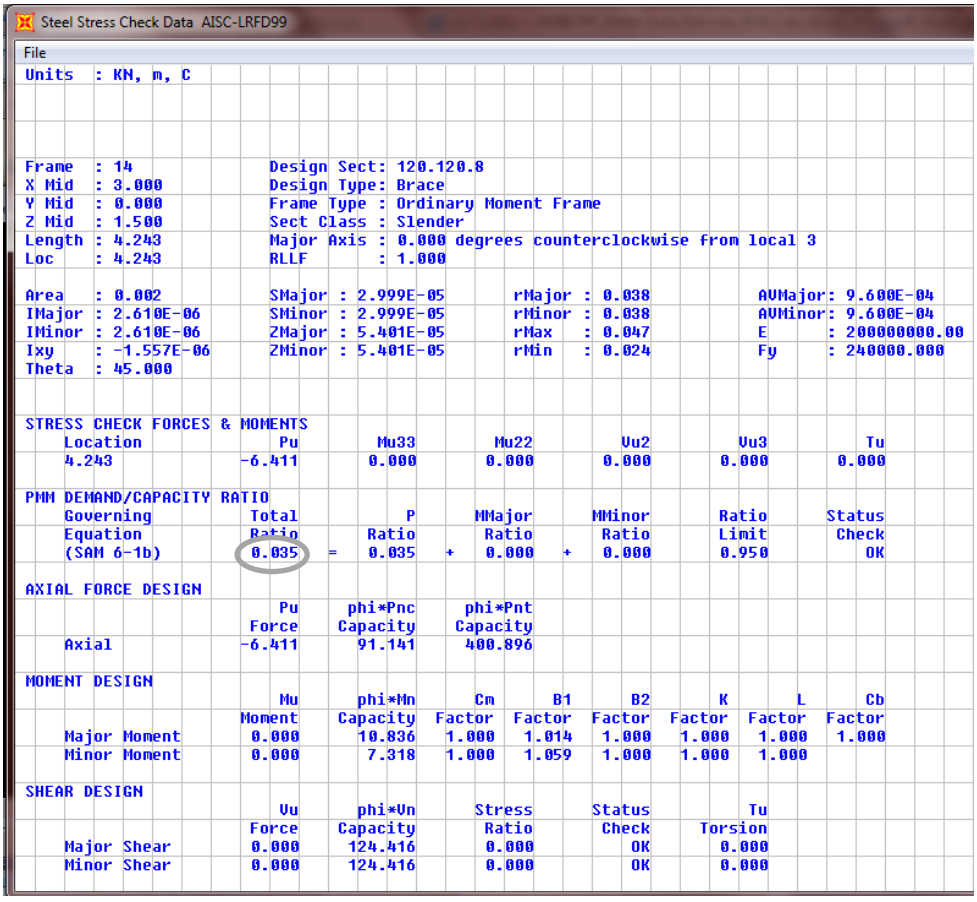
MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	0.000	12.352	1.000	1.000	1.000	1.000	1.000	1.000
Minor Moment	0.000	7.318	1.000	1.000	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	0.000	124.416	0.000	OK	0.000
Minor Shear	0.000	124.416	0.000	OK	0.000

S_6 = - 6.411 kN (tekan)



Steel Stress Check Data AISC-LRFD99

File

Units : KN, m, C

Frame	: 14	Design Sect:	120.120.8		
X Mid	: 3.000	Design Type:	Brace		
Y Mid	: 0.000	Frame Type :	Ordinary Moment Frame		
Z Mid	: 1.500	Sect Class :	Slender		
Length	: 4.243	Major Axis :	0.000 degrees counterclockwise from local 3		
Loc	: 4.243	RLLF :	1.000		
Area	: 0.002	SMajor :	2.999E-05	rMajor :	0.038
IMajor	: 2.610E-06	SMinor :	2.999E-05	rMinor :	0.038
IMinor	: 2.610E-06	ZMajor :	5.401E-05	rMax :	0.047
Ixy	: -1.557E-06	ZMinor :	5.401E-05	rMin :	0.024
Theta	: 45.000				

AVMajor:	9.600E-04										
AVMinor:	9.600E-04										
E	: 200000000.00										
Fy	: 240000.000										

STRESS CHECK FORCES & MOMENTS

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
4.243	-6.411	0.000	0.000	0.000	0.000	0.000

PMM DEMAND/CAPACITY RATIO

Governing Equation	Total Ratio		P Ratio		MMajor Ratio		MMinor Ratio	Ratio Limit	Status Check
(SAM 6-1b)	0.035	=	0.035	+	0.000	+	0.000	0.950	OK

AXIAL FORCE DESIGN

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	-6.411	91.141	400.896

MOMENT DESIGN

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Moment	0.000	10.836	1.000	1.014	1.000	1.000	1.000	1.000
Minor Moment	0.000	7.318	1.000	1.059	1.000	1.000	1.000	

SHEAR DESIGN

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check	Tu Torsion
Major Shear	0.000	124.416	0.000	OK	0.000
Minor Shear	0.000	124.416	0.000	OK	0.000