1020 – Beam in Flexure According to ADM

Description

Verify that a beam of varrying cross-sections made of Alloy 6061-T6 is adequate for the required load, shown in Figure 1, in accordance with the 2020 Aluminium Design Manual [1].

The ADM example references a 6 inch standard pip; parameters are shown in the table below. The results table at the bottom of this document contains more cross-section verification examples.

Material		Modulus of Elasticity	E	10,100.000	ksi
		Yield Strength	F_{ty}	35.000	ksi
		Ultimate Strength	F_{tu}	38.000	ksi
Geometry	Structure	Length	L	10.000	ft
	Cross-section NPS 6"	Diameter	d	6.625	in
		Thickness	t	0.280	in
		Elastic Section Modulus	S	8.500	in
		Plastic Section Modulus	Z	11.300	in
		Radius of Gyration	r_y	1.260	in
		Torsional Constant	J	56.200	in^4
		Moment of Inertia	I_y	28.100	in^4
Load		Dead	P	5.500	kips

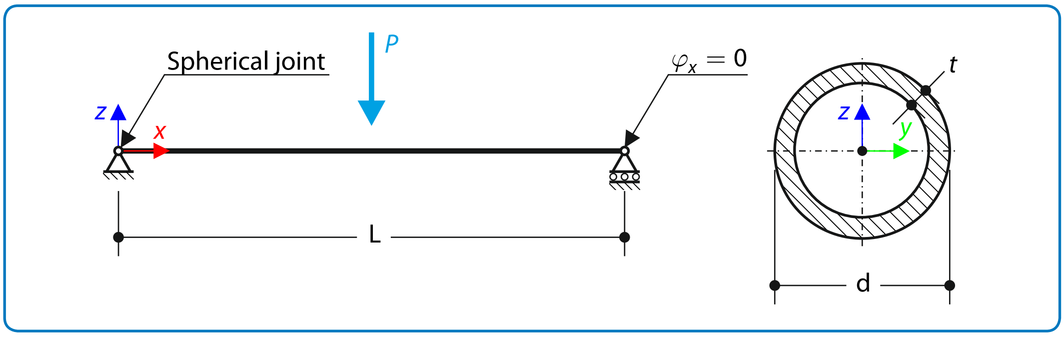

Figure 1: Pipe in flexure

ADM Solution

Section F.1 establishes safety factors of $\Omega_b = 1.95$ on tensile rupture and 1.65 on all other limit states for flexure of building-type structures. The allowable stresses for 6061-T6 given in Part VI Table 2-19 are used below.

Yielding

Section F.2 addresses the limit states of yielding and rupture.

For the limit state of yielding, the allowable moment is the lesser of

$$M_{np}/\Omega_b = \min\{1.5SF_{ty}/\Omega_b,\, ZF_{ty}/\Omega_b\} \approx 239.697 \text{ kip·in.} \tag{1020 – 1}$$

Rupture

For the limit state of rupture, the allowable moment is

$$M_{nu}/\Omega_b = ZF_{tu}/k_t/\Omega_b \approx 220.205 \text{ kip·in,} \tag{1020 – 2}$$

where $k_t = 1$.

Local buckling

The allowable moment for local buckling determined using Section F.3.3 is based on Section B.5.5.4.

$$\sqrt{R_b/t} = \sqrt{\frac{d-t}{2t}} \approx 3.36 < 7.44 = \lambda_1, \tag{1020 – 3}$$

hence

$$F_b/\Omega_b = 39.300 - 2.7R_b/t, \tag{1020 – 4}$$

and the allowable moment for local buckling equals

$$M_{nlb}/\Omega_b = SF_b/\Omega_b \approx 256.799 \text{ kip·in.} \tag{1020 – 5}$$

Lateral-torsional buckling

For closed shapes, the slenderness for lateral-torsional buckling using Section F.4.2.3 is

$$\lambda = 2.3\sqrt{\frac{LS}{C_b\sqrt{I_yJ}}} \approx 11.652 < 66 = C_c \tag{1020 – 6}$$

where $C_b = 1$, therefore, the allowable moment for lateral-torsional buckling equals

$$M_{nmb}/\Omega_b = \frac{M_{np}}{\Omega_b}\left(\left(1 - \frac{\lambda}{C_c}\right) + \frac{\pi^2 E \lambda S}{C_c^3}\right) \approx 218.191 \text{ kip·in} \quad (1020-7)$$

The allowable moment is the least of the allowable moments for yielding **(1020 – 1)**, rupture **(1020 – 2)**, local buckling **(1020 – 5)**, and lateral-torsional buckling **(1020 – 7)**, which is 218.191 kip·in.

From Part VI Beam Formulas Case 1, for a simply supported beam with a concentrated load P at the center, the maximum moment equals

$$M_{max} = PL/4 = 165.000 < 218.191 \text{ kip·in}, \quad (1020-8)$$

therefore, the 6 in schedule 40 pipe is satisfactory.

RFEM 6 Settings

- Modeled in RFEM 6.02.35
- Isotropic linear elastic model is used
- Shear stiffness of members is activated

Results

Structure File	Cross-Section Shape
1020.01	NPS 6×SCH 40
1020.02	Bar 0.500/1.5
1020.03	RT 4×2×0.188
1020.04	I 10×8.65
1020.05	Unsymmetric Beam
1020.06	2.5×2×0.125 Channel, No Stiffeners
1020.06	2.5×2×0.125 Channel, Stiffeners
1020.07	I 8×6.18
1020.08	I 12×14.3, Symmetric
1020.09	I 12×14.3, Unsymmetric
1020.10	RT 3×3×0.095

Shape	RFEM Solution [kip·in]	ADM Solution [kip·in]	Ratio [-]
NPS 6×SCH 40	218.293	218.294	1.014
Bar 0.500/1.5	3.188	3.19	0.999
RT 4×2×0.188	20.461	20.500	0.998
I 10×8.65 (Limiting element)	560.000	560.000	1.000
Unsymmetric Beam	37.869	37.900	0.999
2.5×2×0.125 Channel, No Stiffeners	3.843	3.810	1.009
2.5×2×0.125 Channel, Stiffeners	5.822	5.800	1.004
I 8×6.18	316.061	316.000	0.998
Symmetric I 12×14.3	1151.690	1152.000	1.000
Unsymmetric I 12×14.3	401.565	402.000	0.999
RT 3×3×0.095	17.211	17.100	1.007

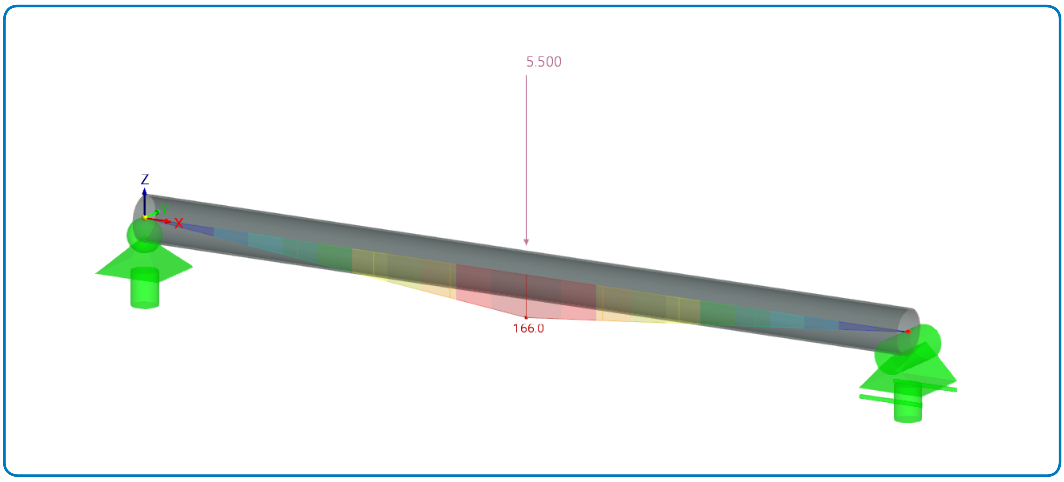

Figure 2: RFEM 6 Results - Bending Moment M_y about Y-axis (Dead Load)

References

[1] THE ALUMINIUM ASSOCIATION, *Aluminium Design Manual*. 2020.