CHAPTER II

LITERATURE REVIEW

AISC *Seismic Provisions for Structural Steel Buildings* (AISC 341-2005) were used to design steel structure with the needs of high ductility level (R> 3). R is the seismic response modification coefficient.

II.1 Load and Resistance Factor Design (LRFD)

In the load and resistance factor design, the design satisfied the requirements if the minimum of design strength is equal to the required strength which determined by the load combination for LRFD calculation.

Design shall be performed in accordance with equation:

$$Ru \leq \varnothing Rn$$

Where: R_u= required strength (LRFD)

R_n= nominal strength

Ø = resistance factor; 0.9 for yielding or compression buckling and 0.75 for rupture (fracture)

ØR_n = design strength

The load combinations are:

1. 1.4 (D+F)
2. 1.2 (D+F+T) + 1.6 (L+H) + 0.5 (L_r or S or R)
3. 1.2D + 1.6 (L_r or S or R) + (0.5L or 0.8W)
4. 1.2D + 1.6W + 0.5L + 0.5 (L_r or S or R)

5. 1.2D + 1.0E + 0.5L + 0.2S
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

Where: D = dead load

E = Earthquake load

F = load due to fluids with well-defined pressures and max heights

H = load due to lateral earth pressure, groundwater pressure, or pressure of bulk materials

L = live load

L_r = roof live load

R = rain load

S = snow load

T = self-straining force

W = wind load

II.2 Compression Members

Compression members are the elements of the structure that only have an axial compressive forces, $f = P/A$, where f is considered become uniform among the entire cross section. One of the compression members is column.

In the AISC specification, the nominal compressive strength is

$$P_n = F_{cr} A_g$$

For LRFD: $P_u \leq Ø_c P_n$

Where: P_u = sum of the factored loads

$Ø_c$= resistance factor for compression = 0.9

$Ø_c P_n$= design compressive strength

The flexural buckling stress, F_{cr}, is determined as follows:

a. When $\frac{KL}{r}$ 4 71 —, F_{cr} = 0 658^{-y}

b. When $\frac{KL}{r}$ 4 71 —, F_{cr} = 0.877 F_e

Where F_e = elastic critical buckling stress

$$F_e = \frac{\pi}{\overline{\ r\ }}$$

II.3 Beams

Beams are the member of the structure that can support transverse loads and mainly subjected to flexure or bending. For flexure, the design moment in the LRFD:

$$M_u \leq Ø_b M_n$$

Where: M_u = required moment strength

$Ø_b$ = resistance factor for bending (flexure) = 0.9

M_n = nominal moment strength

According to the limit states of yielding (plastic moment) and lateral-torsional buckling, the nominal flexural strength, M_n, should be in the lower value. In the yielding:

$$M_y = F_y S_x$$

Where M_y is the bending moment that makes the beam to the yielding value. If $\frac{}{w}$ $3\ 76\ \frac{}{y}$ and $\frac{}{2}$ $0\ 38\ \frac{}{y}$ then shape is compact,

$$M_n = M_p = F_y\ Z$$

Where: F_y = specified minimum yield stress

Z = the x-axis plastic modulus section = $\frac{}{2}$

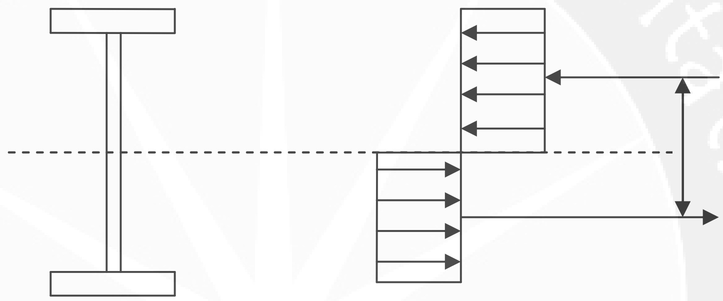

Figure II.1 Compressive and Tensile Stress

$M_n = M_p$ if the $L_b \leq L_p$, if the $L_p < L_b \leq L_r$

$$M_n = \quad 0\ 7 \quad \frac{L \quad L}{L_r \quad L}$$

For $L_b > L_r$

$$M_n = F_{cr}\ S \leq M_p$$

Where

$$F_{cr} = \frac{\pi}{L\ /r\ \ s}\ \ \overline{1 \quad 0\ 078 \frac{}{S} \quad \frac{L}{r\ \ s}}^{\,2}$$

When the shape is non compact, for flange local buckling:

If $\lambda \leq \lambda_p$, there is no flange local buckling

If $\lambda_p < \lambda \leq \lambda_r$, $M_n = \quad 0\ 7 \quad \frac{}{r}$

For lateral-torsional buckling:

If $L_b \leq L_p$, there is no lateral-torsional buckling

If $L_p < L_b \leq L_r$,

$M_n =$ $\quad 0\ 7 \quad \frac{L \quad L}{L_r \quad L}$

If $L_b > L_r$, $M_n = F_{cr}\ S \leq M_p$

Beside the design moment, beam also has shear strength that need to be considered. The nominal strength of shear, V_n, of unstiffened or stiffened webs, according to the limit states of shear yielding and shear buckling, is

$$V_n = 0.6\ F_y\ A_w C_v$$

Where: A_w = area of the web

d = overall depth of the beam

C_v = ratio of critical web stress to shear yield stress

For web of rolled I-shaped members, $C_v = 1.0$ and LRFD $Ø_v = 1.0$, other than it, except round HSS, the C_v is determined:

1. For $h/t_w \leq 1.10 \overline{\quad / \quad}$, $C_v = 1.0$
2. For $1\ 10 \frac{\quad}{y} \quad \frac{\quad}{t_w} \quad 1\ 37 \overline{\quad / \quad}$, $C_v = \frac{1\ 10 \overline{\quad / \ y}}{/t_w}$
3. For $\frac{\quad}{t_w} \quad 1\ 37 \overline{\quad / \quad}$, $C_v = \frac{1\ 51}{/t_w \quad y}$

The relationship between required and available strength is (LRFD)

$$V_u \quad Ø_v V_n$$

Where: $Ø_v$ = resistance factor for shear

V_u = maximum shear based on the controlling combination of factored loads

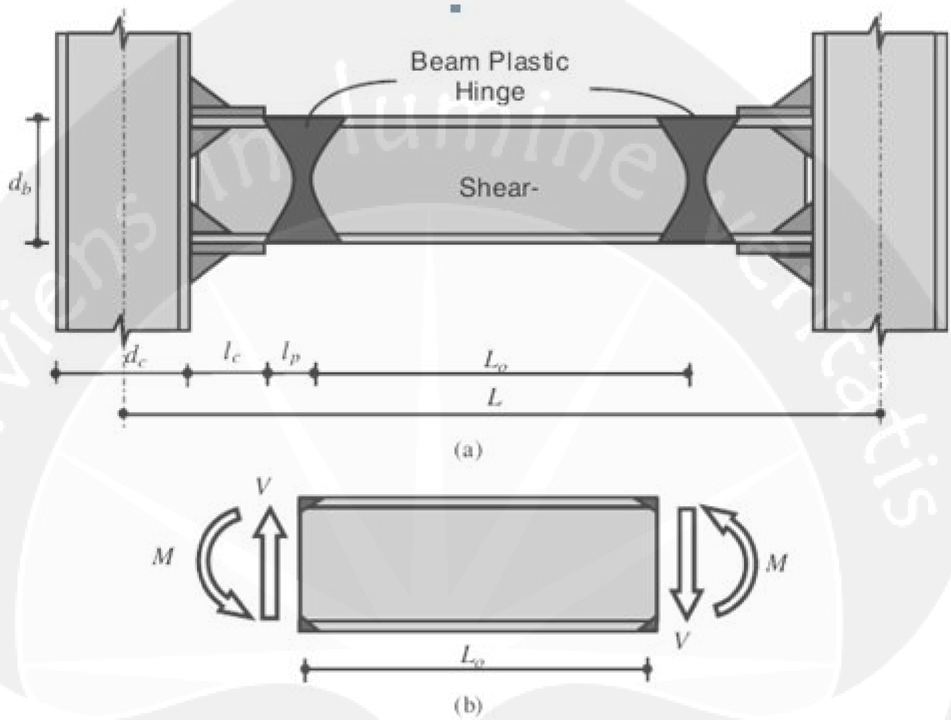

Figure II.2 Location of Plastic Hinge

II.4 Connections

AISC 358-05 set the standard requirement of designing connections for use with Special Moment Frames (SMF) and Intermediate Moment Frames (IMF). Connections designed in AISC 358-05 can be used for structures with LRFD or ASD provisions for the AISC Seismic Provisions.

One type of the connection types from AISC 358-05 is reduced beam section (RBS). Reduced beam section can be used for structures with SMF or IMF system.

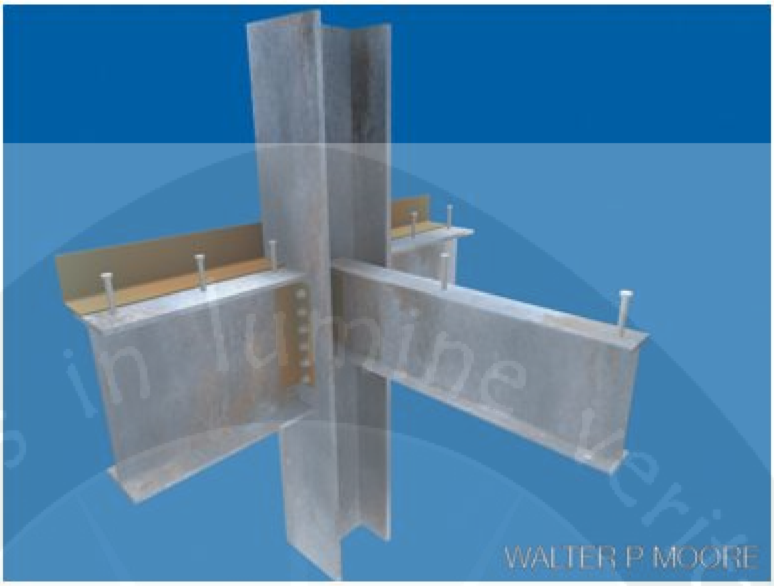

Figure II.3 Steel Beam to Column Connection

II.4.1 Connection Design

It has been mentioned above that to design the connection it should refer to the strength of the bracing, that set as $R_y\ F_y\ A_{g.}$ It means, the strength of the connection should be greater than the bracing connection. it can be written as,

$$P_{connection} > R_y F_y A_g$$

Which is if the connection is designed using bolted or welded, than it should be design to withstand the stress from the bracing.

II.5 Reduced Beam Section

In reduced beam section according to AISC 358-05, portions of the beam flanges are selectively trimmed in the region adjacent to the beam to-column connection. Yielding and hinge formation are intended to occur primarily within the reduced section of the beam.

To design using RBS procedure the beam section, column section and the RBS dimension must be through trial values a, b, and c.

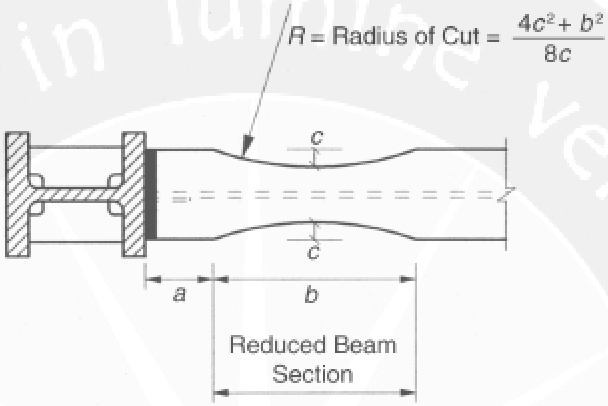

Figure II.4 RBS Dimensions

The limitation to the trial values as follows:

$0.5\ b_{bf}$ a $0.75\ b_{bf}$

0.65 d b 0.85 d

$0.1\ b_{bf}$ c $0.25\ b_{bf}$

The plastic modulus will be at the center of reduced beam section. ($Z_e = Z_x - 2ct_{bf}\,(d - t_{bf}\,)$).

Maximum probable moment at the center of the reduced beam section can be determine by r . The values of C_{pr} shall not be exceed than 1.2 and the values of Ry can refer to tablel 1-6-1 of AISC 341-05.

The shear force at the center of the reduced beam sections shall be determined by a free body diagram of the portion of the beam between the centers of the reduced beam sections. This calculation shall assume the moment at the center of each reduced beam section is M_{pr} and shall include gravity loads acting on the beam based on the load combination $1.2D + f1L + 0.2S$.

Moment at the face of the column is computed as $M_f = M_{pr} + V_{RBS}\,S_h$. The equation neglects the gravity load.

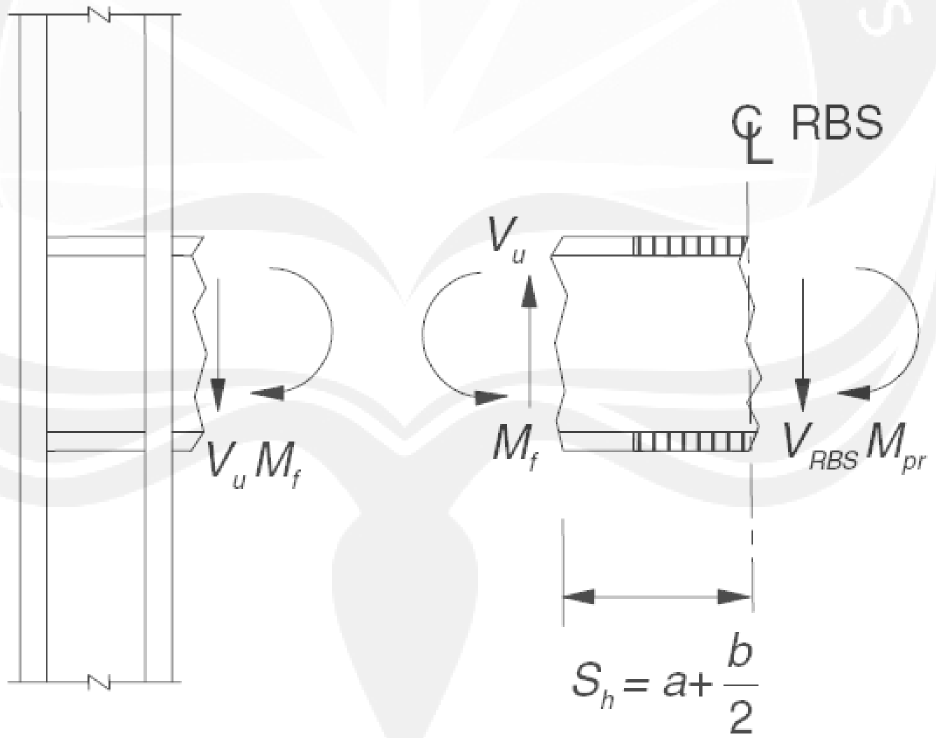

Figure II.5 Free-body diagram between center of RBS and face of column

Plastic moment can be calculated by $M_{pe} = Z_b R_y F_y$. The plastic moment should not bigger than moment at the face of the column (M_f).

The required strength *Vu* will be calculated as follows $\frac{2 \quad r}{L}$

$gra \quad it$ The design of shear strength of the beam refers to AISC Specifications chapter G.

To check the column-beam moment ratio $\sum \qquad r$. M_v is $RBS \quad \frac{}{2} \quad \frac{}{2}$.

II.6 Shear Influence in Beam Moment Capacity

Current design code specifications for design of beam-to-column connections do not consider the effect of reduction in maximum developable moment capacity in the beam, due to the presence of shear. This may result in the heavier connections. (Jaswant and Murty, 2004) A fiber model is employed to obtain the V-M interaction curves of thirteen AISC W-sections (namely W36×300, W33×240, W27×177, W21×142, W24×160, W18×114, W16×96, W14×426,W14×84, W12×190, W12×58, W10×112, and W8×67) (AISC, 1989). The cross-section is discretized into fibers of 1 mm thickness that are parallel to the major axis of bending. The dimensions of the beam section are rounded-off to the nearest millimeter.

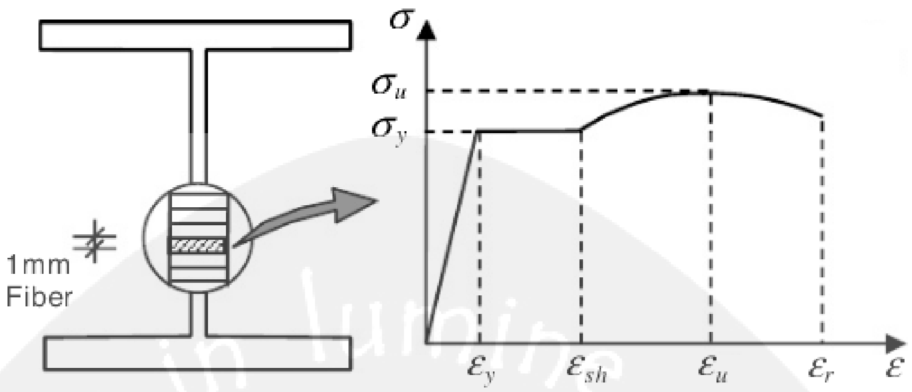

Figure II.6 Fiber model of W-sections: (a) Discretisation of the beam section across the cross-section, and (b) Explicit form of stress-strain curve of steel (Murty and Hall, 1994).

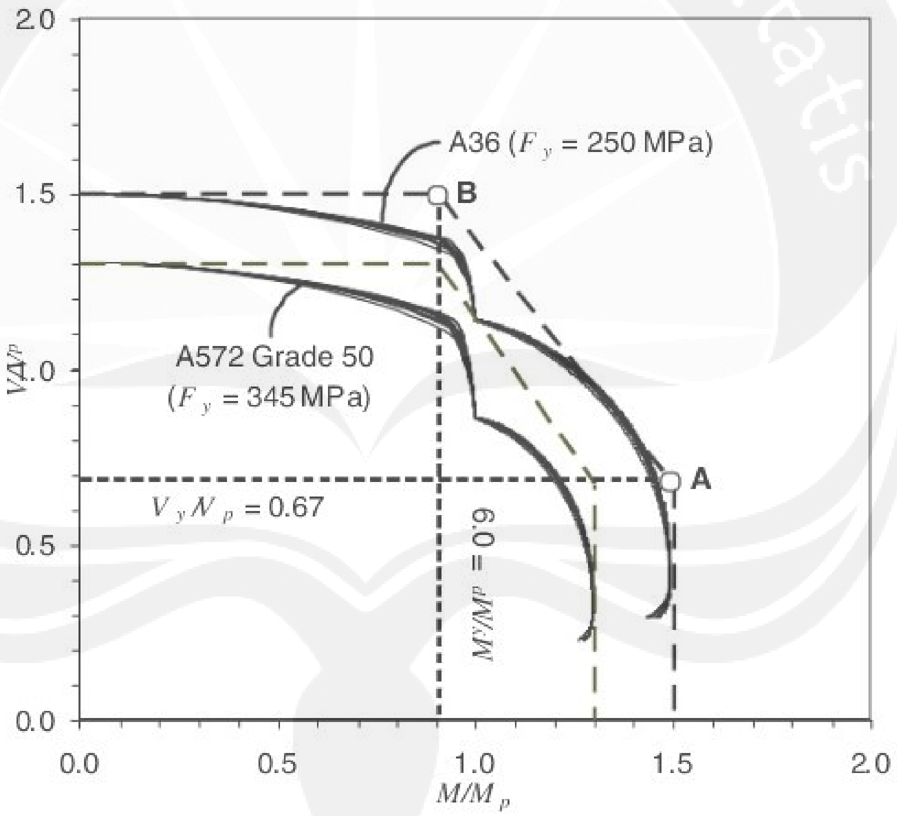

Figure II.7 Normalized V-M interaction surfaces for AISC W-sections with idealized upper bound forminimum specified yield strength and F_u/F_y = 1.5 (F_y = 250 MPa) and F_u/F_y = 1.3 (Fy = 345 MPa), and R_y=1.0

The design of beam-to-column connections as per capacity design concepts requires that the connections be designed for the maximum moment and shear that are expected to be developed in the beam. In the existing method for design of beams, a section with M_p larger than the maximum bending moment demand M is selected. It is then ensured that V_p of the section is larger than the maximum shear demand V.

Table II.1 Limiting L/d values for the AISC W-sections used to develop the V-M interaction curves

Sr. No.	W-Section	d_b mm	t_w mm	S_b $\times10^{-4}m^3$	Z_b $\times10^{-4}m^3$	$\frac{M_p}{V_p}$ m	A36 (F_u/F_y=1.5) $\frac{L_{oA}}{d_b}$	A36 (F_u/F_y=1.5) $\frac{L_{oB}}{d_b}$	A572 Gr 50 (F_u/F_y=1.3) $\frac{L_{oA}}{d_b}$	A572 Gr 50 (F_u/F_y=1.3) $\frac{L_{oB}}{d_b}$
1	W36×300	933	24	181.1	205.7	1.59	7.67	2.00	6.65	2.31
2	W33×240	851	21	132.9	150.5	1.46	7.71	2.02	6.68	2.33
3	W27×177	694	18	80.8	91.3	1.27	8.20	2.15	7.11	2.48
4	W21×142	545	17	52.0	58.5	1.09	9.03	2.38	7.83	2.74
5	W24×160	628	17	67.8	76.0	1.23	8.83	2.33	7.66	2.69
6	W18×114	469	15	36.1	40.6	1.00	9.60	2.52	8.32	2.91
7	W16×96	415	14	27.2	30.5	0.91	9.85	2.61	8.54	3.01
8	W14×426	475	48	115.9	142.5	1.08	10.25	2.47	8.89	2.85
9	W14×84	360	11	21.5	23.8	1.04	13.03	3.47	11.29	4.01
10	W12×190	365	27	43.1	51.0	0.90	11.06	2.77	9.59	3.20
11	W12×58	310	9	12.8	14.2	0.88	12.77	3.42	11.07	3.94
12	W10×112	289	19	20.7	24.2	0.76	11.87	3.01	10.29	3.48
13	W8×67	229	15	9.9	11.5	0.58	11.38	2.91	9.86	3.35

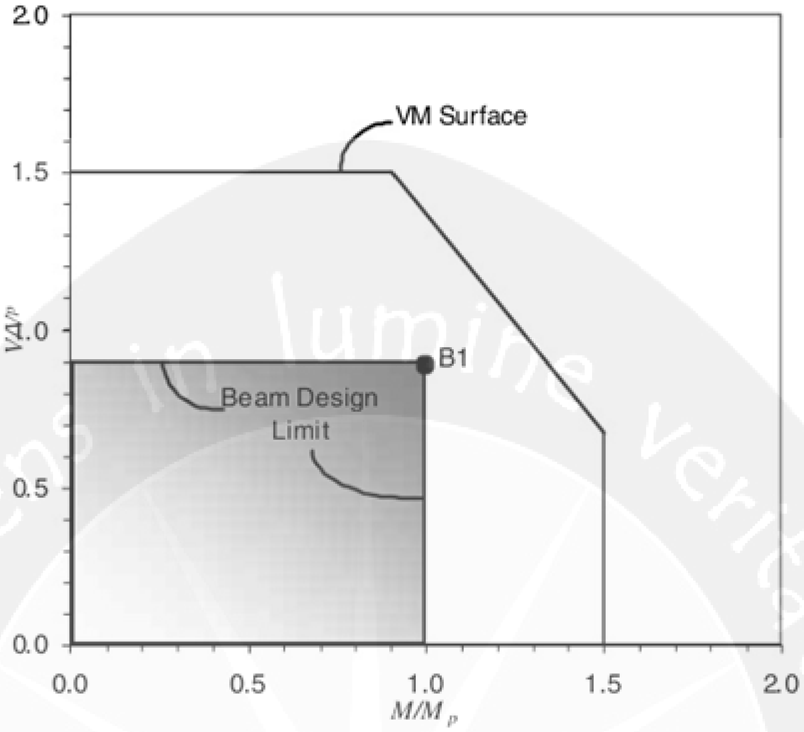

Figure II.8 Limiting V-M interaction curve for design of beam to column connections with minimum specified yield strength and Fu/Fy = 1.5 (for A36 steel) compared with the design limits employed in the existing beam design procedures.