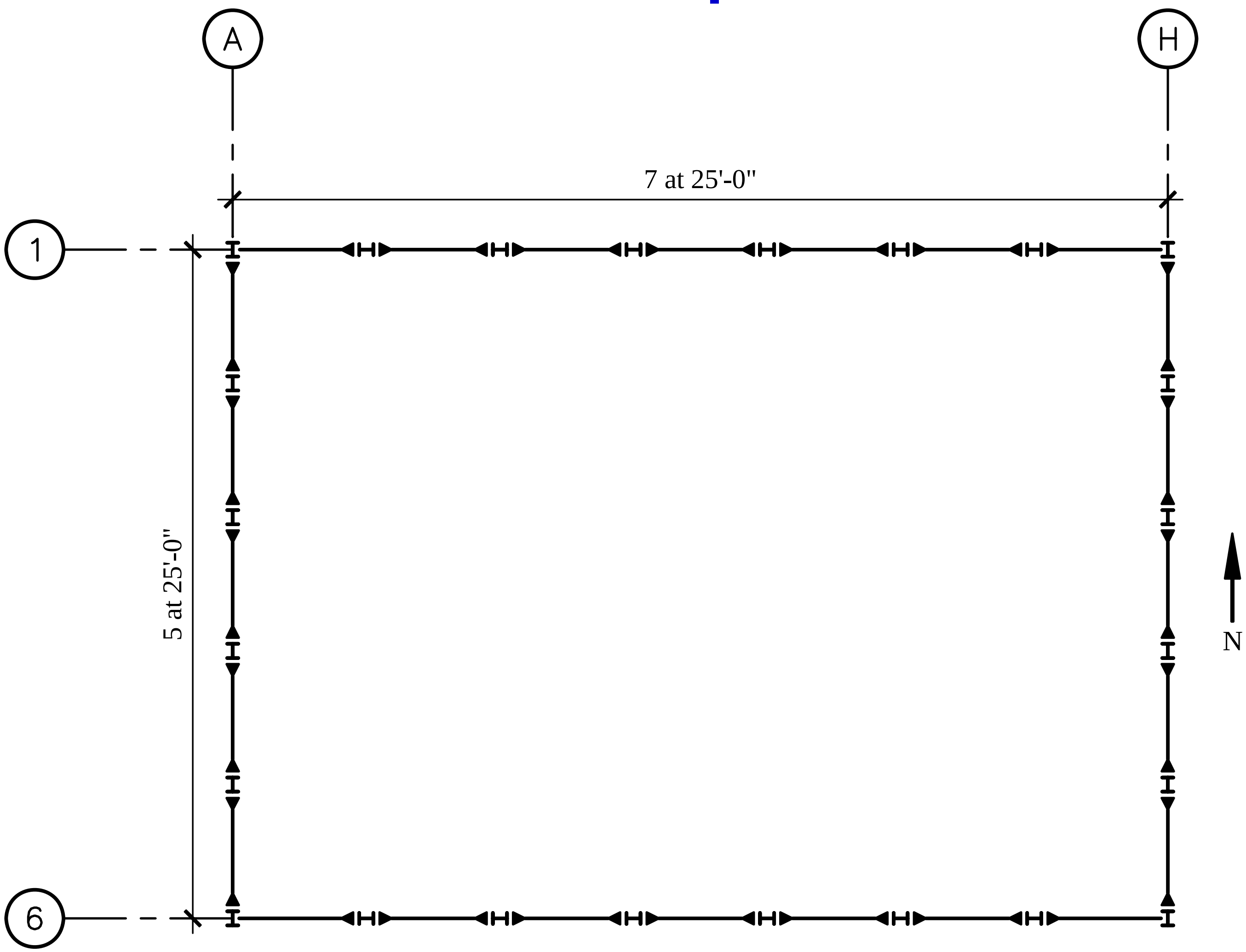
Special Moment Frames

Example



Special Moment Frames

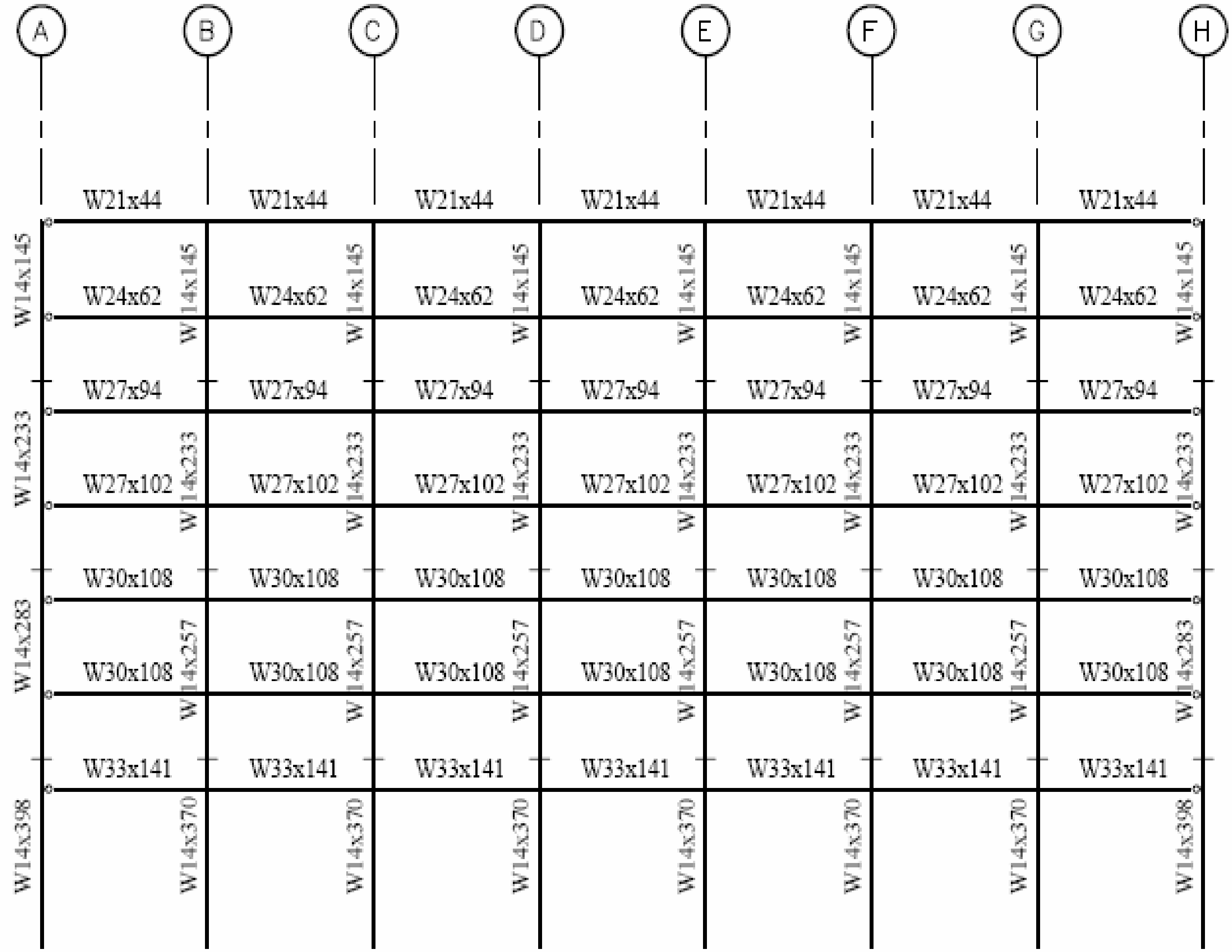
The following design steps will be reviewed:

- Select preliminary member sizes
- Check member local stability
- Check deflection and drift
- Check torsional amplification
- Check the column-beam moment ratio rule
- Check shear requirement at panel zone
- Select connection configuration

Special Moment Frames

Select preliminary member sizes – The preliminary member sizes are given in the next slide for the frame in the East-West direction. These members were selected based on the use of a 3D stiffness model in the program RAMFRAME. As will be discussed in a subsequent slide, the drift requirements controlled the design of these members.

SMF Example – Preliminary Member Sizes



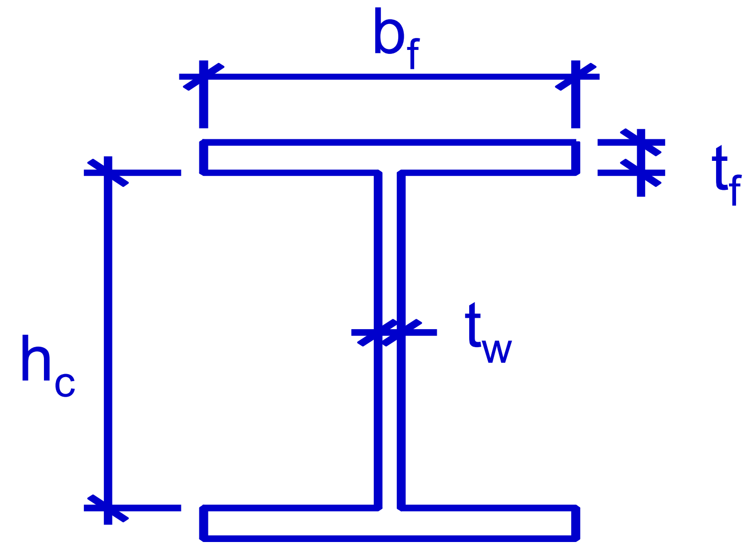
SMF Example – Check Member Local Stability

Check beam flange: $\dfrac{b_f}{2t_f} = 6.01$
(W33x141 A992)

Upper limit: $0.3\sqrt{\dfrac{E}{F_y}} = 7.22$ OK

Check beam web: $\dfrac{h_c}{t_w} = 49.6$

Upper limit: $3.76\sqrt{\dfrac{E}{F_y}} = 90.6$ OK



SMF Example – Check Deflection and Drift

The frame was checked for an allowable story drift limit of $0.020h_{sx.}$ All stories in the building met the limit. Note that the *NEHRP Recommended Provisions* Sec. 4.3.2.3 requires the following check for vertical irregularity:

$$\frac{C_d \Delta_{x\ \ story\ 2}}{C_d \Delta_{x\ \ story\ 3}} = \frac{\left(\dfrac{5.17 in.}{268 in.}\right)}{\left(\dfrac{3.14 in.}{160 in.}\right)} = 0.98 < 1.3$$

Therefore, there is no vertical irregularity.

SMF Example – Check Torsional Amplification

The torsional amplification factor is given below. If Ax < 1.0 then torsional amplification is not required. From the expression it is apparent that if $\delta_{max} / \delta_{avg}$ is less than 1.2, then torsional amplification will not be required.

$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{avg}} \right)^2$$

The 3D analysis results, as shown in FEMA 451, indicate that none of the $\delta_{max} / \delta_{avg}$ ratios exceed 1.2; therefore, there is no torsional amplification.

SMF Example – Member Design NEHRP Guide

Member Design Considerations - Because $P_u/\varphi P_n$ is typically less than 0.4 for the columns, combinations involving Ω_0 factors do not come into play for the special steel moment frames (re: AISC Seismic Sec. 8.3). In sizing columns (and beams) for strength one should satisfy the most severe value from interaction equations. However, the frame in this example is controlled by drift. So, with both strength and drift requirements satisfied, we will check the column-beam moment ratio and the panel zone shear.

SMF Example – Column-Beam Moment Ratio

Per AISC Seismic Sec. 9.6

$$\frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0$$

where ΣM^*_{pc} = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. ΣM^*_{pc} is determined by summing the projections of the nominal flexural strengths of the columns above and below the joint to the beam centerline with a reduction for the axial force in the column.

ΣM^*_{pb} = the sum of the moments in the beams at the intersection of the beam and column centerlines.

SMF Example – Column-Beam Moment Ratio

Column – W14x370; beam – W33x141

$$\Sigma M_{pc}^{*} = \Sigma Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right) = 2\left[736 in^2 \left(50 ksi - \frac{500 kips}{109 in^2} \right) \right]$$

$$\Sigma M_{pc}^{*} = 66,850 in - kips$$

Adjust this by the ratio of average story height to average clear height between beams.

$$\Sigma M_{pc}^{*} = 66,850 in - kips \left(\frac{268 in. + 160 in.}{251.35 in. + 128.44 in.} \right) = 75,300 in - kips$$

SMF Example – Column-Beam Moment Ratio

For beams:

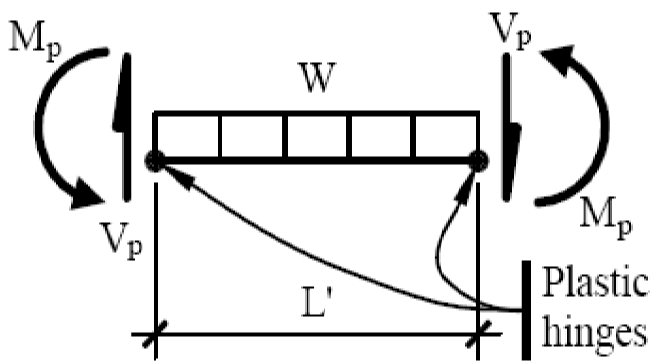
$$\Sigma M^{*}_{pb} = \Sigma(1.1 R_y M_p + M_v)$$

$$\text{with } M_v = V_p S_h$$

$$S_h = \text{dist. from col. centerline to plastic hinge}$$

$$= d_c/2 + d_b/2 = 25.61 in.$$

$$V_p = \text{shear at plastic hinge location}$$



$$V_p = \left[2M_p + \left(wL'^2/2\right)\right] = \frac{2M_p + \frac{wL'^2}{2}}{L'}$$

$$= \frac{(2)(25{,}700 in - kips) + \left(\frac{(1.046 klf)}{12}\frac{(248.8 in.)^2}{2}\right)}{248.8 in.} = 221.2 kips$$

SMF Example – Column-Beam Moment Ratio

$$M_v = V_p S_h = (221.2 kips)(25.61 in.) = 5{,}665 in - kips$$

and

$$\Sigma M_{pb}^{*} = \Sigma(1.1 R_y M_p + M_v)$$

$$= 2[(1.1)(1.1)(25{,}700 in - kips) + 5{,}665 in - kips] = 73{,}500 in - kips$$

The ratio of column moment strengths to beam moment strengths is computed as:

$$Ratio = \frac{\Sigma M_{pc}^{*}}{\Sigma M_{pb}^{*}} = \frac{75{,}300 in - kips}{73{,}500 in - kips} = 1.02 > 1.00 \quad \therefore OK$$

Other ratios are also computed to be greater than 1.0

SMF Example –Panel Zone Check

The 2005 AISC Seismic specification is used to check the panel zone strength. Note that FEMA 350 contains a different methodology, but only the most recent AISC provisions will be used. From analysis shown in the NEHRP *Design Examples* volume (FEMA 451), the factored strength that the panel zone at Story 2 of the frame in the EW direction must resist is 1,883 kips.

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] = (0.6)(50ksi)(17.92in.)(t_p)\left[1 + \frac{(3)(16.475in.)(2.66)^2}{(33.3in.)(17.92in.)(t_p)}\right]$$

$$R_v = 537.6t_p + 315$$

The required total (web plus doubler plate) thickness is determined *by*:

$$\phi R_v = R_u$$

$$(1.0)(537.6t_p + 315) = 1{,}883 kips$$

$$t_{p_{required}} = 2.91 in.$$

The column web thickness is 1.66*in*., *therefore the required doubler plate thickness is*:

$$t_{p_{doubler}} = 1.25in. \quad (\textit{therefore use one } 1.25in. \textit{ plate or two } 0.625in. \textit{ plates})$$

SMF Example – Connection Configuration

Beam-to-column connections used in the *seismic load resisting system* (SLRS) shall satisfy the following three requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 radians.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

(3) The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_yM_p]/L_h \qquad (9\text{-}1)$$

SMF Example – Connection Configuration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 9.2a by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Appendix P.

(c) Provision of qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

  (i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix S.

  (ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

Special Moment Frames
Summary

Beam to column connection capacity

Select preliminary member sizes

Check member local stability

Check deflection and drift

Check torsional amplification

Check the column-beam moment ratio rule

Check shear requirement at panel zone

Select connection configuration

- Prequalified connections
- Testing