RULES OF THUMB FOR STEEL DESIGN

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ABSTRACT: In earlier times when computers were neither available nor essential, one objective of the structural design process was to discover a computational method, which was elegant, simple and appropriately accurate. When such a process was identified it was recorded as an expedient approach to solving a recurring structural design problem. Thus, quick "Rules of Thumb" became essential resources for the structural engineer. As computer software has proliferated, become very comprehensive, and been made very user friendly, the importance of "Rules of Thumb" and approximate methods has been diminished. It has been argued that, with the computational speed and ease of application of computer methods, the need for approximations and "Rules of Thumb" no longer exists. However, equally imposing arguments can be made for the value of these quick approaches such as:

- The structural engineer should have tools to make on-the-spot intelligent decisions,
- A reasonable solution is often required as computer input,
- The validity of the computer output should be verified with rational approximations.

So, with the objective of fostering continued development, use and enthusiasm for "Rules of Thumb" and approximate methods, several steel framing "Rules of Thumb" are presented in this paper. Because a majority of these have been developed over the past they are based on Allowable Stress Design (ASD). Future development will be based on Load and Resistance Factor Design (LRFD).

STRUCTURAL DEPTHS: An appropriate question often raised in a project initiation meeting is what will be the structural depth? This information is integral to setting the floor to floor dimension. Therefore, it is important to have established "Rules of Thumb", to facilitate structural depth predictions. The depth of the structural system is influenced by the span of the elements as well as such variables as the spacing of elements, loads and loading conditions, continuity, etc. Nonetheless, ratios of span to depth can often be relied upon to provide a guide and a starting point from which further refinement can be made. With the caution that variables other than span need to be considered, the following guide is presented:

SYSTEM	L/d_s	SPAN RANGE
Steel Beam	20 to 28	0' to 75'
Steel Joist Floor Member Roof Member	 20 24	8' to 144'
Plate Girder	15	40' to 100'
Joist Girder	12	20' to 100'
Steel Truss	12	40' to 300'
Space Frame	12 to 20	80' to 300'

Figure 1

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It is convenient to remember that serviceable steel section depths are in the range of ½” of depth for each foot of span (i.e. L/24). Some might find it easier to remember the following simplified rules where the length is expressed in feet and the depth of the member in inches:

Depth of Roof Beams, Roof Joists = 0.5*Length
Depth of Floor Beams, Floor Joists = 0.6*Length
Depth of Composite Beams = 0.55*Length

An alternate span/depth rule for composite beams is to use a ratio of L/21 for the structure depth from the top of the slab to the bottom of the beam. For example an 18” deep composite beam supporting a 5 ½” slab should be serviceable for a span of 41’ (i.e. 41 x 12 / 23 ½ ≈ 21).

<u>SECTION PROPERTIES</u>: The section properties of a wide flange cross section can be approximated with reasonable accuracy using the transfer term of the moment of inertia equation (parallel axis theorem) and some reasonable assumptions for the center of gravity of parts of the cross section. The schematic cross section in Figure 2 indicates an assumed center of gravity for the upper and lower “T” sections.

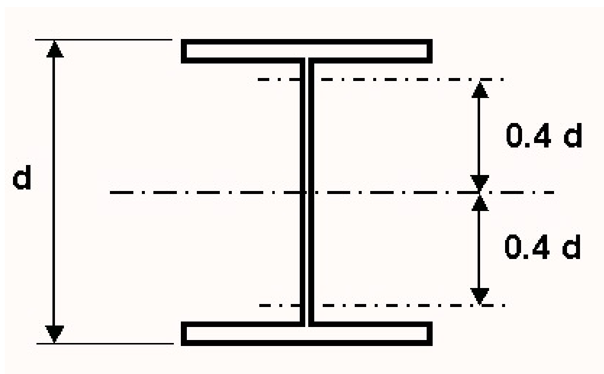

Figure 2

The moment of inertia can be calculated assuming the section consists of two T sections.

$$\mathbf{I = \Sigma\left(I_0 + A\, y^2\right)}$$

If $A = A_s/2$, where A_s is the total area of the section, and $y = 0.4d$ and only the transfer component of the moment of inertia equation is considered then:

$$\mathbf{I \approx A_s\ 0.16\, d^2}$$

The density of steel is 490 pcf. Therefore, the cross sectional area of a member can be converted to weight by multiplying the area by 3.4 (i.e. 490/144).

$$\mathbf{I \approx \frac{Wt\, h^2}{21.27}}$$

The moment of inertia will be larger than that predicted by the preceding equation, since the I_0 terms has been neglected. Therefore, using 20 in the denominator will improve the approximation. The resulting equation is a surprisingly accurate approximation for the moment of inertia of a steel wide flange section.

BEAM MOMENT OF INERTIA

$$\boxed{\mathbf{I \approx \frac{Wt\ d^2}{20}}}$$

An approximate equation for the section modulus is obtained by dividing the moment of inertia by d/2 resulting in the following equation:

BEAM SECTION MODULUS

$$\boxed{\mathbf{S \approx \frac{Wt\ d}{10}}}$$

The radius of gyration is defined as $\sqrt{I/A}$. The flanges have the dominant contribution to the moment of inertia in the weak axis. The radius of gyration can be approximated as $\sqrt{\frac{2\ t_f\ b^3}{12} \Big/ 2\ t_f\ b} = \sqrt{\frac{b^2}{12}}$. This approximation suggests $r_y \approx 0.29$ b. The more easily remembered relationship of 0.25 b has been found to be an accurate approximation to the weak axis radius of gyration.

WEAK AXIS
BEAM RADIUS OF GYRATION

$$\boxed{\mathbf{r_y \approx 25\%\ b}}$$

By observation, the radius of gyration about the strong axis can be approximated as a function of the member depth as follows:

STRONG AXIS
BEAM RADIUS OF GYRATION

$$\boxed{\mathbf{r_x \approx 45\%\ d}}$$

FLEXURAL CAPACITY: If the allowable stress is set at 0.67 Fy, a convenient equation for determining the foot weight required to satisfy a moment capacity for a specific member depth can be derived using the approximate section modulus. For Fy = 36 ksi, Fb = 24 ksi and a rapid determination of a steel section size can be made without reference to a steel manual using a very simple equation.

BEAM WEIGHT

$$\boxed{Wt \approx \frac{5\ M}{D}} \quad \textbf{Fy = 36 ksi}$$

The closest economy section of the depth used in the equation that has a foot weight greater than predicted by the equation indicates the beam that will sustain the moment.

Consider a beam spanning 30 feet supporting a 10 foot width of floor with a total supported load of 140 psf, resulting in a moment of 157.5 foot-kips. For an 18" deep beam, the equation yields 43.75 pounds per foot. A W18x50 is the predicted section and the actual moment capacity is 176 foot-kips. If a beam depth of 21" is assumed, the equation yields 37.5 suggesting a W21x44, which has a moment capacity of 162 foot-kips.

A similar formulation for steel having ***Fy = 50*** ksi produces:

BEAM WEIGHT $$\boxed{Wt \approx \frac{3.5\ M}{D}}$$ **Fy = 50 ksi**

For an 18" deep beam, the equation yields 30.6 pounds per foot, therefore, a W18x35 is predicted. The actual capacity of a W18x35 beam with Fy = 50 ksi is 158 foot kips.

For common composite beam floor systems (e.g. 5½" slabs with 3" composite deck, 4½" slab with 2" composite deck, etc.), the simplified equations yield relatively accurate foot weights if 70% to 75% of the simple span moment is used for M.

SHEAR STUDS: "Rules of Thumb" can often be determined by observation. The following are two more "Rules of Thumb" relating to composite construction and ***Fy=36 ksi:***

> ***In ASD Number of shear studs required for Full Composite Action = 1.1*Wt***
> ***In LRFD Number of shear studs required for Full Composite Action = 1.25*Wt***

COLUMNS: Simple column capacity equations can be derived by observing the graph of the column capacity as a function of KL/r. The light curved line in Figure 3 is the column curve for steel having a yield strength of 36 ksi. An estimate of the allowable column stress can be made by drawing a linear approximation to the curve. This approach is used in lieu of performing linear regression to arrive at an easily remembered equation for allowable stress as a function of KL/r.

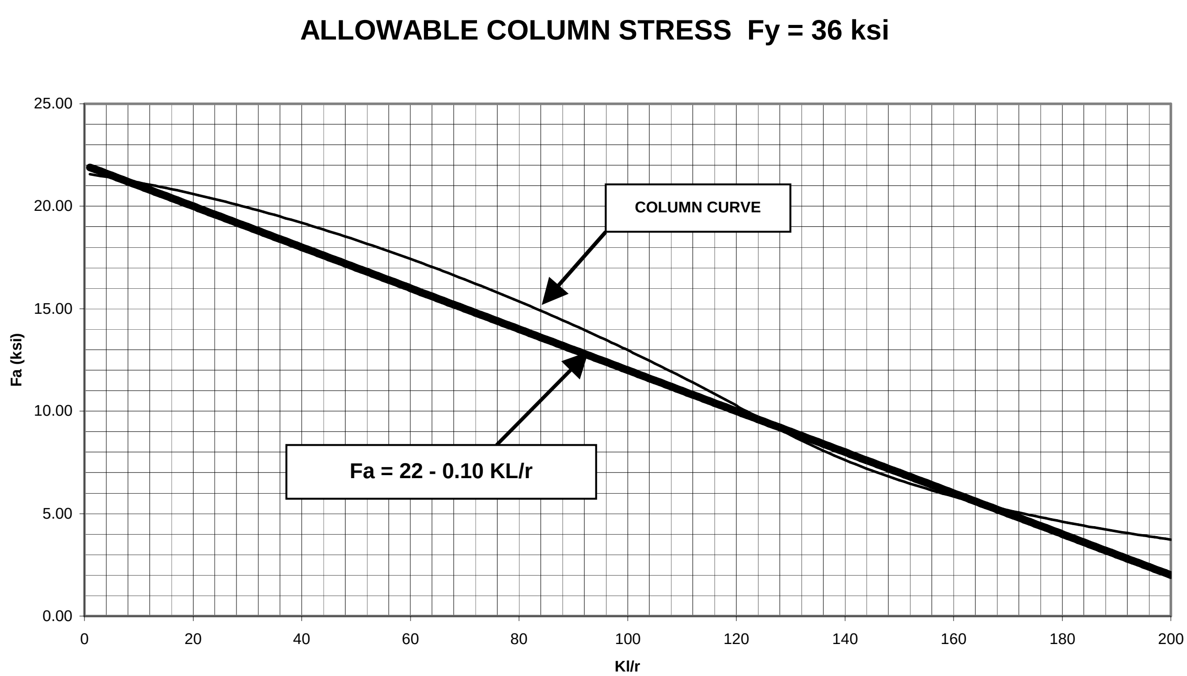

Figure 3

Thus, for Fy = 36 ksi the column capacity can be predicted by:

COLUMN CAPACITY

$$\boxed{P_a \approx A\left(22 - 0.10\,{}^{KL}\!/_{r}\right)}$$

Fy = 36 ksi

A similar relationship can be determined for steel having a yield strength of 50 ksi:

COLUMN CAPACITY

$$\boxed{P_a \approx A\left(30 - 0.15\,{}^{KL}\!/_{r}\right)}$$

Fy = 50 ksi

<u>ROOF SYSTEMS</u>: A common approach to economy in steel roof systems of single story buildings is to cantilever girders over the columns. The ends of the cantilever support a reduced span beam. When this system is subjected to a uniform load and multiple equal spans are available, a cantilever length approximately equal to 15% (0.146) of the span length will result in the maximum moment in any span being equal to 1/16 wL^2. For end spans, negative and positive moments can be balanced using a cantilever length equal to 25% of the first interior span.

Another approach to economical roof systems is the use of plastic analysis. Although not as critical for this system, splice locations in the plastically designed continuous beams are usually chosen so that they are close to the point of zero moment.

> ***Hinge or splice location for cantilever or continuous roof systems is 15% to 25% of span length***

<u>TRUSSES</u>: The foot weight of trusses utilizing ***Fy=36*** ksi steel can be calculated by assuming Fa=22 ksi. The Chord Force (F_{ch}) is then equal to the moment (M) in foot-kips divided by d_e (the effective depth – lever arm) in feet, resulting in a chord area of $M/22d_e$. By recognizing that ***Wt = A*3.4***, converting d_e to inches and assuming that $d_e = 0.9D$ and that the total truss weight is equal to 3.5 times the chord weight then:

TRUSS WEIGHT

$$\boxed{\text{Truss Wt} \approx \frac{6\,M}{d}}$$

Fy = 36 ksi

If the steel yield strength is 50 ksi the following approximation results:

TRUSS WEIGHT

$$\boxed{\text{Truss Wt} \approx \frac{4.5\,M}{d}}$$

Fy = 50 ksi

These approximations are sufficient to include connection material.

<u>RIGID FRAME ANALYSIS APPROXIMATIONS</u>: The following “Rules of Thumb” are useful in determining preliminary sizes for Rigid Moment Frames resisting Lateral loads. They are based on the traditional “Portal Frame” approach modified from the authors’ experiences with “real” frames.

$$M_{col} \approx \frac{1.2H}{2} \bullet \frac{V_{story}}{n_{col}}$$

$$M_{beam} \approx {M_{col}}/{2} \quad \textit{Interior Columns at Roof}$$

$$M_{beam} \approx M_{col} \quad \textit{Interior Columns Not at Roof}$$

The moments in beams framing into exterior columns are half of the above values

STEEL WEIGHT ESTIMATES: Cost is generally the basis for confirming a structural system since safety and function are essential for any options considered. Economy is related to the weight of the structural steel although costs are influenced by many other parameters. Yet, weight can be a valuable indicator of cost and "Rules of Thumb" are useful in establishing an expectation for steel weight. A quick assessment of anticipated weight serves as a check of the reliability of the weight determined by more involved investigations.

A linear approximation to the weight of the structural steel elements can be determined by plotting steel weights as a function of stories.

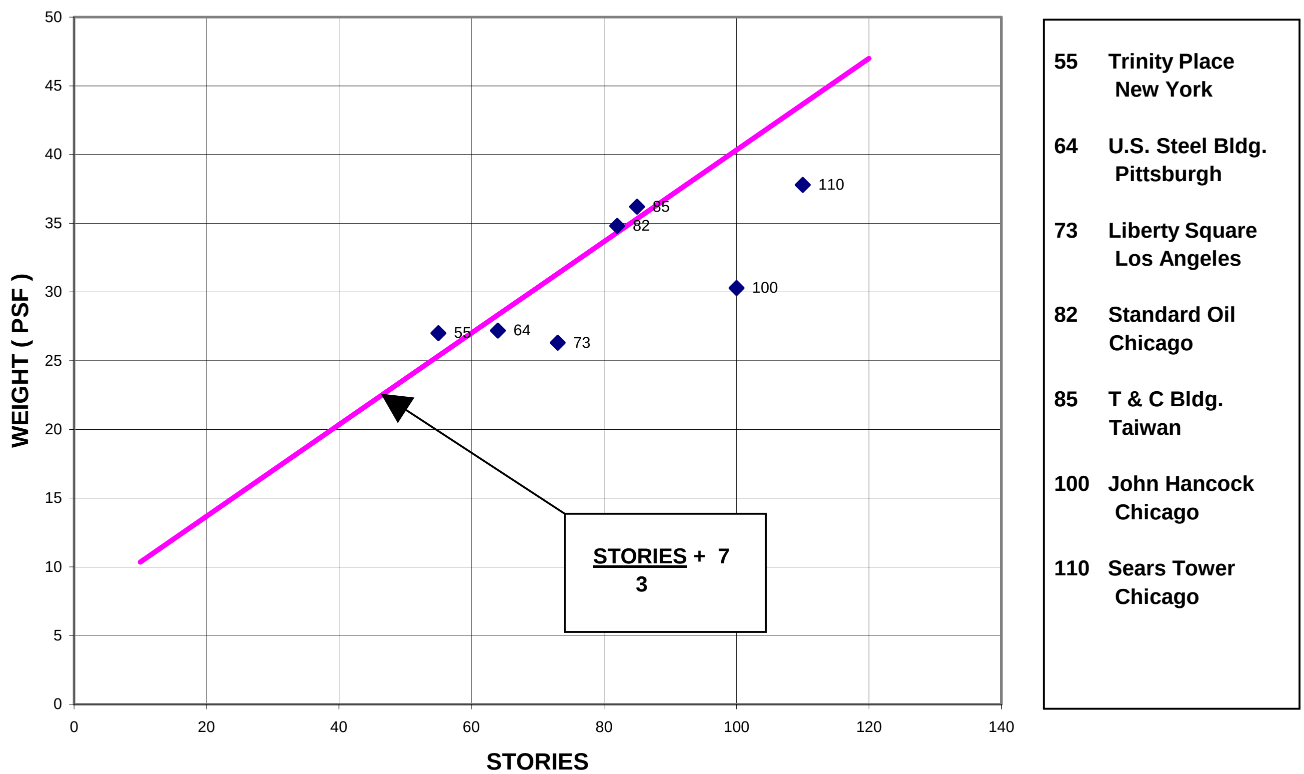

Figure 4

The line defined by **STORIES/3 + 7** is a reasonable approximation for the structure weight.

STEEL WEIGHT $\boxed{\textbf{Wt(psf) = stories/3 + 7}}$

A three-story building would have a steel weight in the range of 8 psf and a 27-story building would require 16 psf. Certainly, this relationship is an over simplification. Yet, it provides a value, which can be used to confirm that the results of a more detailed analysis are reasonable.

TALL BUILDING STRUCTURAL SYSTEMS: The late Fazlur Khan hypothesized that the appropriate structural system to resist lateral loads was directly related to building height. He predicted that structural economy could be realized using the appropriate system as follows:

STORIES	LATERAL LOAD RESISTING SYSTEM
<30	Rigid frame
30 to 40	Frame – shear truss
41 to 60	Belt truss
61 to 80	Framed tube
81 to 100	Truss – tube w/ interior columns
101 to 110	Bundled tube
111 to 140	Truss – tube without interior columns

MISCELLENEOUS:

End rotation of a simple beam = 0.2 radians
Deflection of simple span beam (reduction due to connections) = 80% of calculated
Roof Framing Systems

For Cantilevered or continuous roof beams :

- **Run beams in short direction**
- **Optimum bay size is 30' x 40'**

For Truss Joist and Joist roof systems:

- **Run Girders in Long direction**
- **Optimum bay size is 40' x 40'**

NOMENCLATURE:

A = Area (in^2)
D = Nominal member depth (inches)
d_s = System depth (ft)
Fy = Yield strength of steel
H = Story Height
I = Moment of Inertia (in^4)
L = Length (ft)
M = Bending moment (foot-kips)
M_{beam} = Design Moment for Beam
M_{col} = Design Moment for Column
n_{col} = Number of Columns (not bays) in the story of the Frame
S = Elastic Section Modulus (in^3)
V_{story} = Total Story Shear for the Frame
Wt = Foot weight of the steel beam (pounds per foot)
Wt(psf) = Weight of steel structure (psf)