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December 8, 1988

Mr. Geerhard Haaijer
American Institute of Steel Construction
Wrigley Building
400 North Michigan Ave., 8th Floor
Chicago, IL 60611

Subject: LRFD Study

Dear Mr. Haaijer:

First, I would like to introduce myself. I am a structural engineer and partner with THP Limited in Cincinnati. We are a medium-sized structural engineering firm and do a wide variety of building and restoration work.

We have been very interested in AISC's new LRFD specification, but locally there has been little impetus to use it. The courses AISC has sponsored in the area have been good LRFD introductions, but they are not really sufficient with which to start full-scale designing.

I took the opportunity of completing a Master's degree to study LRFD from a consultant's viewpoint. This was not only to compare the component design by LRFD and ASD but to try to explain why they are different or similar.

A summary of the thesis's conclusions include:

1. LRFD is more rational than ASD and I plan to continue using it.

2. The design philosophy of steel controlled by strength (yield or buckling) has not greatly changed even though the equations look strange to us. The major design change has occurred with composite beams, everything else being somewhat similar to ASD.

3. The major differences in sizes of components designed by LRFD vs. ASD have resulted (with the exception of composite beams) from the ANSI load factors/combinations. My research has shown that:

   a. The ASD/LRFD calibration has been done at a higher live load to dead load ratio than is normal for practical design office work.

b. The LRFD equations can substantially reduce the impact of the 33% wind-increase in allowable stresses permitted by ASD.

The conclusions noted in items #2 and #3 have not been stressed to the design profession. I do not know if this is intentional or if no one has used LRFD enough to appreciate the impact.

As the closing point of the thesis, I compared the ANSI LRFD gravity load combination equations with what I believe is a more rational load combination equation (see Figure 20, page 72). This, I believe, deserves some discussion from the profession.

I would appreciate any comments you or your office have regarding my conclusions. During the preparation of this thesis, I discussed my concerns with Dr. Galambos, Bill Liddy, and Bob Lorenz. Bill Liddy suggested I send this study to you for review. I believe designers would be interested in a practical study of LRFD and a better explanation of why results with LRFD vary from previous ASD experience.

Thanks very much for your time.

Very truly yours,

THP LIMITED
CONSULTING ENGINEERS

Shayne O Manning

Shayne O Manning, P.E., S.E.

SOM:bar

Enclosure

cc: Bill Liddy (AISC)

1378

A STUDY OF THE

LOAD AND RESISTANCE FACTOR DESIGN METHODOLOGY

A thesis submitted to the

Division of Graduate Studies and Research
of the University of Cincinnati

in partial fulfillment of the
requirements for the degree of

MASTER OF SCIENCE

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of the College of Engineering

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by

Shayne O Manning

BSCE, University of Cincinnati, 1977

ABSTRACT

This thesis examines the American Institute of Steel Construction's Load and Resistance Factor Design Specification for Structural Steel Buildings and compares it to the previous AISC specification which was based upon allowable stress design criteria.

The thesis contrasts these two specifications by the comparison of load factors and effects on components' design with graphs and figures and also by presenting the design of three structures by both LRFD and ASD.

The conclusion addresses the impact of the load factor methodology on design office practice and suggests a different load factor equation for gravity-controlled design.

ACKNOWLEDGMENTS

I would like to express my appreciation to the following people for their assistance in completing this thesis:

Dr. Frank Weisgerber, for his guidance and critical review of this study;

Alleen, my wife, for her patience and positive attitude throughout this period;

My parents, who have always been an incentive to me to do my very best;

Jill Toelke and Betty Rossi, for their uncomplaining labor in typing of the many revisions to this thesis;

Last and certainly not least, to Louis T. Tallarico, my mentor, for without his excellent and kind example I would not be where I am today.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

AISC	American Institute of Steel Construction
ASD	Allowable Stress Design (specifically AISC Specification 8th Edition 1978 - Ref. 3).
A_c	Area of concrete slab within effective width.
A_s	Area of steel cross-section.
A_w	Area of member's web.
B_1, B_2	Factors used to determine second order M_u from first order analysis.
b	Compression element width.
C_b	Bending coefficient dependent upon moment gradient.
C_m	Coefficient applicable to bending term in beam column equation, directly related to column curvature characteristics
C_c	Column slenderness separating elastic and inelastic buckling F_{cr} (at C_c) = $F_y/2$
d	Depth of member
DL	Dead loads - weight of permanent construction including fixed service equipment (Ref. 4).
FR	Fully restrained connections, i.e., full moment capacity rigid connection with negligible rotations due to connecting material.
F_a	Axial design strength.
F_{cr}	Critical axial strength.
F_y	Minimum yield stress.
f'_c	specified compressive strength of concrete.
k	Effective length factor for prismatic member
l	Unbraced length of member.
LF	Load factor - see also γ
LL	Live loads - produced by use and occupancy of building (including partitions) and do not include environmental loads or dead loads. (Ref. 4).

LRFD	Load and Resistance Factor Design (specifically AISC Specification 1st Edition 1986-Ref. 2).
L_b	Laterally unbraced length, length between points which are either braced against lateral displacement of compression flange or twist of the cross-section.
L_c	Maximum unbraced length at which allowable bending strength = 0.66 F_y (ASD).
L_p	Maximum unbraced length for beam to attain full plastic bending moment.
L_{pd}	Maximum unbraced length for beam to attain full plastic bending moment and a minimum of 3 rotations required for plastic analysis.
L_r	Unsupported length separating inelastic and elastic lateral-torsional buckling.
L_u	Maximum unbraced length at which allowable bending strength = 0.60 F_y (ASD)
M_{cr}	Elastic buckling moment
M_p	Plastic bending moment
M_r	Buckling moment at L_r
M_u	Required bending moment strength
N	Total number of headed 3/4" diameter shear studs welded to steel member for composite action.
PR	Partially restrained connections (semi-rigid) which have a moment capacity dependent upon joint rotation. Includes almost all "pinned" connections
P_e	Euler buckling strength
P_f	Probability of the limit state being exceeded for structural usefulness.
P_u	Required axial strength
Q	Loading
R	Component Resistance (usually strength of component but can also be a serviceability characteristic).
r	Radius of gyration.
r_t	Radius of gyration of compression flange plus one third of compression portion of the web taken about an axis in the plane of the web.

S	Elastic section modulus
SF	Shape Factor = Z/S
SL	Snow load specified by appropriate building code.
t_w	thickness of web.
V	Shear force.
V_u	Required shear strength
WL	Wind load specified by appropriate building code.
Z	Plastic section modulus
ϕ	Resistance factor ≤ 1.0
β	Safety or reliability index
γ	Load factor > 1.0

CHAPTER I

INTRODUCTION

The purpose of this study is to compare the typical component design methodologies of the two structural steel building specifications enforced in this country at the present time and to examine differences which may result from using one specification as opposed to the other.

AISC's Allowable Stress Design (ASD), 8th edition, and Load and Resistance Factor Design (LRFD), 1st edition, are contrasted for the component designs of columns, beams, and composite beams. Graphs and figures are presented to describe influences of live load to dead load ratios, load combinations with wind, and revisions in design philosophy.

A dual design (ASD and LRFD) is shown for three distinctly different buildings: an arena, a four-story office building, and a twenty-one story office building. The design results have been tabulated and compared on figures to illustrate the similarities and differences between the two specifications.

The case studies are concluded with an explanation of the differences between the resulting component sizes with particular reference to the individual member-type design comparisons given in the previous chapters.

The conclusions of this study comparing LRFD and ASD usage in the design office are:

1. LRFD gives a more economical structure and therefore a lower factor of safety for the majority of gravity load controlled designs encountered in everyday office practice.

2. LRFD gives a less economical and correspondingly a higher factor of safety for the majority of wind load controlled designs encountered in everyday office practice.

3. LRFD challenges the designer more with respect that:

   a. More thought (and time) is needed to be given to establishing the proper loading and combinations thereof.
   b. Many design procedures are more time consuming and therefore are counterproductive to design efficiency.
   c. Serviceability criteria such as deflection, vibration, and ponding control final designs in more cases since LRFD has a substantial strength capacity over ASD.

The final conclusion of this study is that the process of establishing LRFD as the structural steel design tool is an ongoing task. This is the structural steel state-of-the-art design procedure but the writer believes there will be modification of the various load and resistance factors in this specification prior to the design profession generally adopting LRFD.

CHAPTER II

STRUCTURAL STEEL DESIGN SPECIFICATIONS' BACKGROUND

Structural steel has historically been designed by allowable stress procedures. This method assumes an elastic stress distribution based upon compatibility of strains and compares the calculated service load stresses to a code established allowable stress. This allowable stress is an ultimate failure stress divided by an appropriate factor of safety.

The American Institute of Steel Construction has published the Specification For the Design Fabrication and Erection of Structural Steel for Buildings since 1923 and it is now in its eighth edition with the allowable stress design procedures forming the main focus of the code. The specification has included a section (Part 2) since 1956 which includes design procedures for members analyzed by limit state methods, i.e. plastic analysis.

In recent years, several other countries have adopted a strength design procedure for steel design. AASHTO has an alternative strength design procedure with The Load Factor Design for Bridges and ACI is almost exclusively load factor design with ACI 318 Strength Design for Reinforced Concrete.

AISC published their first comprehensive non-allowable stress design specification in 1986. This is based upon a probabilistic set of load and resistance factors and is the culmination of more than fifteen years of study and calibration.

CHAPTER III

LRFD DESIGN

LRFD FORMAT

LRFD separates the capacity of the structure from the loading superimposed upon it. This is very similar to the approach ACI took with their "strength" design criteria.

The theoritical capacity (or strength) of the structural component is reduced by a factor ϕ (sometimes known as the undercapacity factor in concrete design). The applied loading is increased by the load factor γ . The combined application of these factors is to insure an appropriate level of safety for the member. The generic LRFD equation, which is proposed to be used for all construction materials' strength design, including that for structural steel, is:

$$\phi R \geq \sum \gamma Q$$

ϕ - Dimensionless resistance factor $\leq$ 1.0 reflecting uncertainties in material properties, and manner and consequences of failure.

R - Nominal resistance of component or section based upon geometrical and material properties (i.e. strength).

γ - Load factor corresponding with various types of loading to account for deviations in loading and analysis procedures.

Q - Loading function and type live load, dead load, wind load, etc. and axial, bending, shear.

R and Q are representative of strength and load in whatever design-based comparison is appropriate for the particular situation. For example, loads can be stress, externally applied forces, or internally generated forces such as that due to restraint or settlement and strength will be the comparable resistance to this load.

Development of the LRFD Criteria

The goal of LRFD is to obtain a more uniform factor of safety than is possible with ASD for all loading conditions. The measure of factor of safety used is the safety or reliability index β .

The relationship between the loading and the component strength can be shown by frequency distributions (see Fig. 1). Any overlap between Q and R will result in an unsatisfactory state, i.e. no safety margin of the applied load being above the limit state (strength or serviceability criteria of the member).

Two facts should be noted from this graph (Fig. 1):

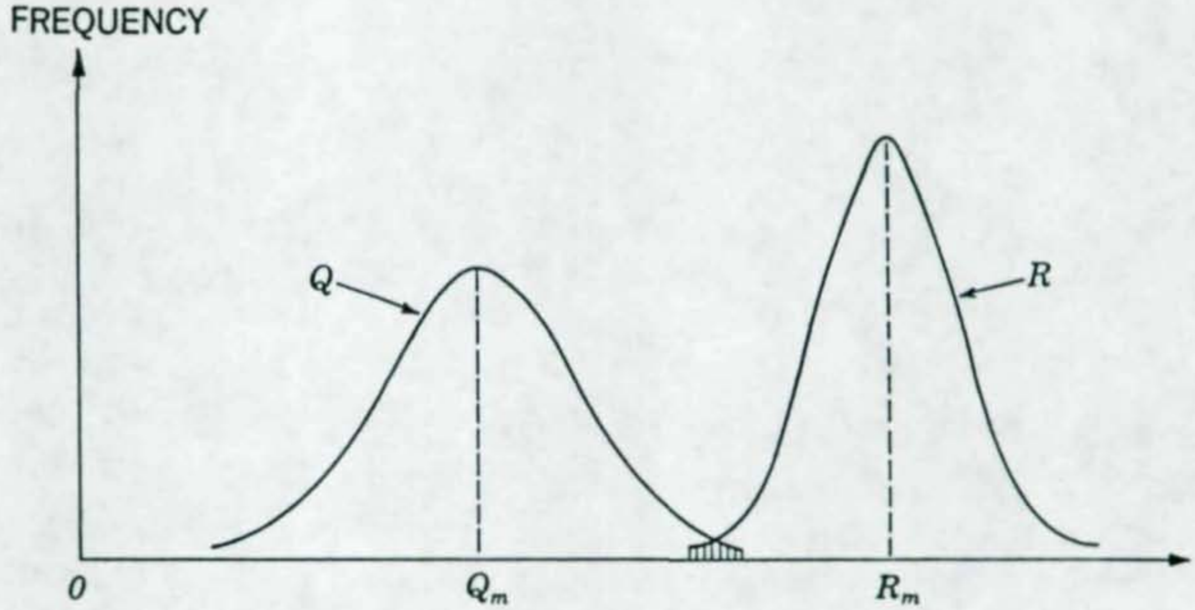


Figure 1 Frequency Distribution of Load Effect Q and Resistance R
(Ref. 2)

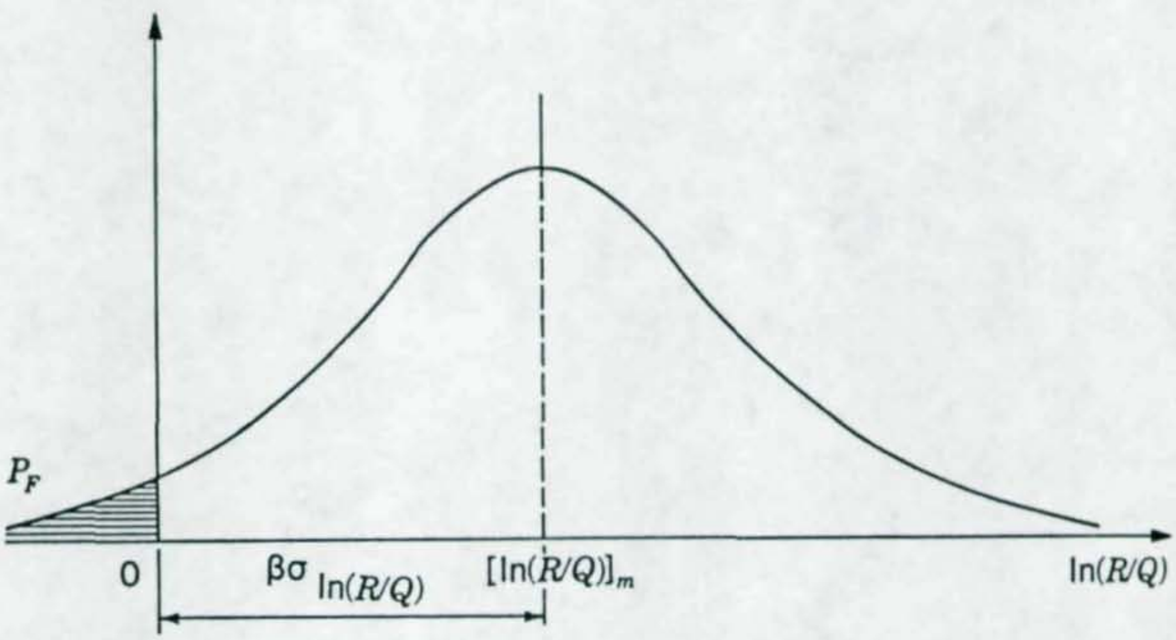


Figure 2 Definition of Reliability Index β
(Ref. 2)

1. It is virtually impossible to eliminate all possibility that $Q > R$.

2. The smaller the overlapping shaded area, the larger the safety margin against the load exceeding the appropriate limit state.

It has been found that a more easily understood method to calculate and visualize this measure of safety β is to plot the frequency distribution of the log of the strength divided by the load ($\ln(R/Q)$) (see Fig. 2).

In simplistic form, the safety index β is the number of standard deviations of the probability of the mean of the function of $\ln (R/Q)$ to the right of the absissca. Any probability to the left of the absissca, has an unacceptable safety factor on the corresponding limit state.

The safety index β can be determined by a probabilistic analysis using the mean and standard deviations of the various parameters affecting the strength and load (Ref. 14). As can be seen from Fig. 2, for any given relationship between load and strength, β is directly related to the limit state probability P_f. It was in this manner that β was used to make adjustments to the load and resistance factors to calibrate ASD and LRFD specifications (Refs. 12 and 16).

The development of LRFD for structural steel was a two stage process. There did not exist an adequate basis of appropriate load factors to use with a strength design and there was not a good understanding of the existing safety indices produced by present ASD design.

Safety factors are needed to account for variations in both strength and load. Not only does the load vary in intensity and our ability to predict it, and material and geometric properties have a certain variance, but also our analysis procedures can be overly simplistic in terms of the actual load distribution within the structure. As noted earlier, the committee developing the LRFD specification used the β index to measure the safety resulting from present and proposed designs since it can account for the variance for both load and strength as well as the variance in the accuracy of the analytic model.

The 1982 edition of ANSI A58.1 was the vehicle which presented the generic load factor/limit design methodology to the design profession. The intent was that though the load factors were directly aimed at a LRFD steel specification, the ϕ factors for other materials such as concrete, wood, masonry, etc., could be calibrated by their own industry/specification writing groups to convert all materials to a consistent strength design.

Developing the load factors was not possible without also considering the resulting ϕ factors. To make sense to the practitioner, load factors by definition need to be greater than 1.0 and resistance factors should be less than 1.0. What becomes obvious from the literature is that ϕ factors are sub-routines of the specified load factors.

LRFD uses the specified load factors to account for variations in loading, and uses the resistance factors, not as normally thought of based upon the properties of the physical system, but to account for the ductility of a failure and more importantly to calibrate the resulting design strength to present design practice by using the safety index, β .

LRFD Load Factors

LRFD load factors were developed by an ANSI committee and the most common load combinations are noted in the LRFD specification. The load combinations which were used by the writer for the design comparisons (see Chapter VII) include:

1.4 DL

1.2 DL + 1.6 LL + 0.5 SL

1.2 DL + 0.5 LL + 1.6 SL

1.2 DL + 0.5 (LL + SL) + 1.3 WL

0.9 DL - 1.3 WL

These load combinations are based upon the very small probability of the maximum lifetime live load occurring simultaneously with the maximum lifetime snow and/or wind loads. The load combinations were developed in combination with appropriate ϕ factors to give a β safety index of (Ref. 2):

3.0 for gravity loads

4.5 for connections

2.5 for combinations of gravity and wind load

Calibrating the γ and ϕ factors to current ASD component designed was based upon a LL/DL ratio = 3.0 for most LRFD components (Ref. 15). In fact, when current ASD components were analyzed for their β safety index, there was found an inconsistent and large variance in most instances (Ref. 16).

Figures 3 and 4 are graphs of average resulting load factors vs. ratios of live load, dead load and wind load. Several facts should be noted from these figures:

1. A LL/DL ratio of 3 or greater gives a LF = 1.5 or just slightly larger.
2. A LL/DL ratio of 0.125 or smaller is controlled by "1.4 DL."
3. A LL/DL ratio between 0 and 1.0 has a LF between 1.24 and 1.4.

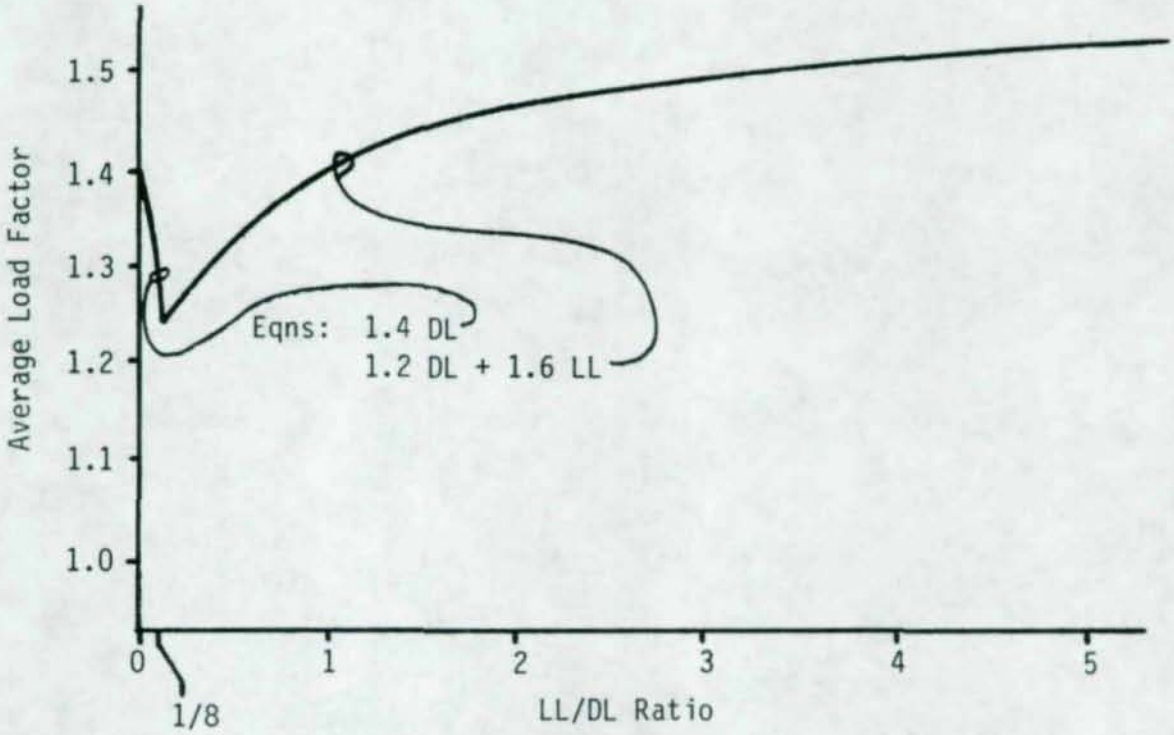


Figure 3 Load Factor Study (Gravity Loads)

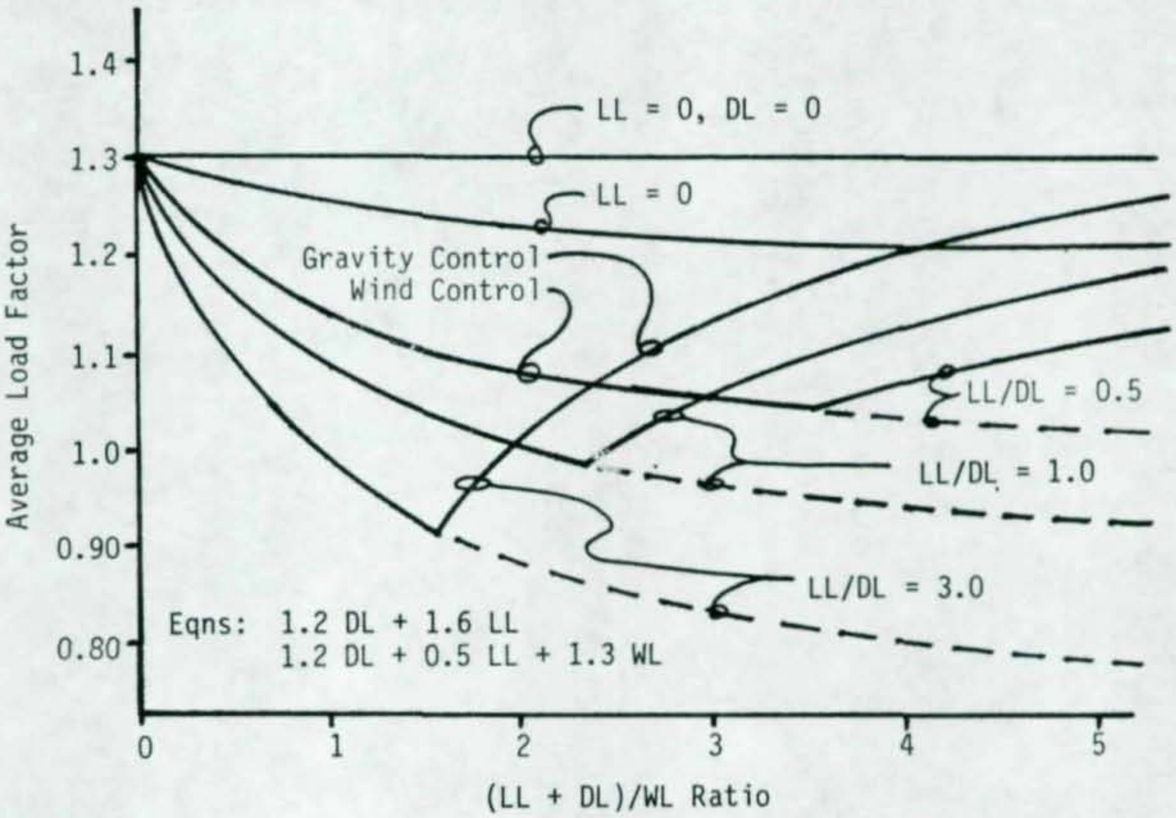


Figure 4 Load Factor Study (Wind and Gravity Loads)

4. A component with WL and no LL or DL has a LF of 1.3.
5. Higher percentages of LL combined with WL will give smaller average load factors.

One fact which has direct bearing on this discussion is the ANSI A58.1-82 provision regarding ASD load combinations. (Ref. 4). This does not specifically permit the 33% increase on allowable stresses historically used by the profession, if only wind load generated forces are present. AISC ASD 8th Edition does unequivocally permit this increase in allowable stresses for wind-generated forces acting alone or in concert with other gravity loads. (Ref. 3).

Figures 3 and 4 will be referred to later in this thesis as explanations for differences in ASD and LRFD designed components.

CHAPTER IV

COLUMN DESIGN BY LRFD

Specification Review

Column Design has undergone a distinct revision from ASD to LRFD. The AISC ASD inelastic column design has been based upon the tangent modulus criteria since its 6th Edition (Ref. 7) and LRFD inelastic column design is based upon a maximum strength criteria.

The maximum strength column design approach takes advantage of the unyielded portions of the column cross-section (the portion of the cross-section with compressive residual stresses reaches yield first) to stabilize the column, but also considers the initial crookedness of the member. This procedure was not practical until the 70's when the computer became available to perform the iterative solution.

The tangent modulus column design approach considers residual stresses in much the same manner as the maximum strength approach except this is done with the closed form solution of a differential equation and also initial crookedness is accounted for with a varying factor of safety. The factor of safety chosen was 1.67 ($kl/r = 0$) varying to 1.92 ($kl/r \geq Cc$). The rational behind this sliding scale was that initial crookedness effects are most pronounced at intermediate slenderness ratios (Ref. 7).

The elastic portion of both ASD and LRFD column design curves is still based upon the theory proven by Euler in 1744.

The easiest way to show the column equations' differences between ASD and LRFD is on a plot of allowable capacity or stresses vs. slenderness ratio. Figure 5 plots service load allowable stresses versus kl/r. Three facts can be seen from this figure:

1. Stocky columns (kl/r between 0 to 60) have substantially more capacity at LL/DL ratios between 0 to 1.0 with LRFD procedures.
2. LRFD has no provision for increased allowable stresses for secondary members at higher slenderness ratios. Reportedly this is also to be taken out of the AISC ASD's 9th Edition (Ref. 15).
3. LRFD column curves have been calibrated to ASD from about the midpoint of the intermediate slenderness range through the Euler buckling range with a LL/DL ratio of approximately 1.1 (Ref. 2).

The inelastic buckling portion of the graph details the approximately 15% increase in permitted service load stresses possible with stocky columns and a load factor of 1.3. It can be shown that columns in multistory buildings will incorporate this 15%± savings. The code permitted live load reduction (i.e. 100 psf is reduced to 40 psf for

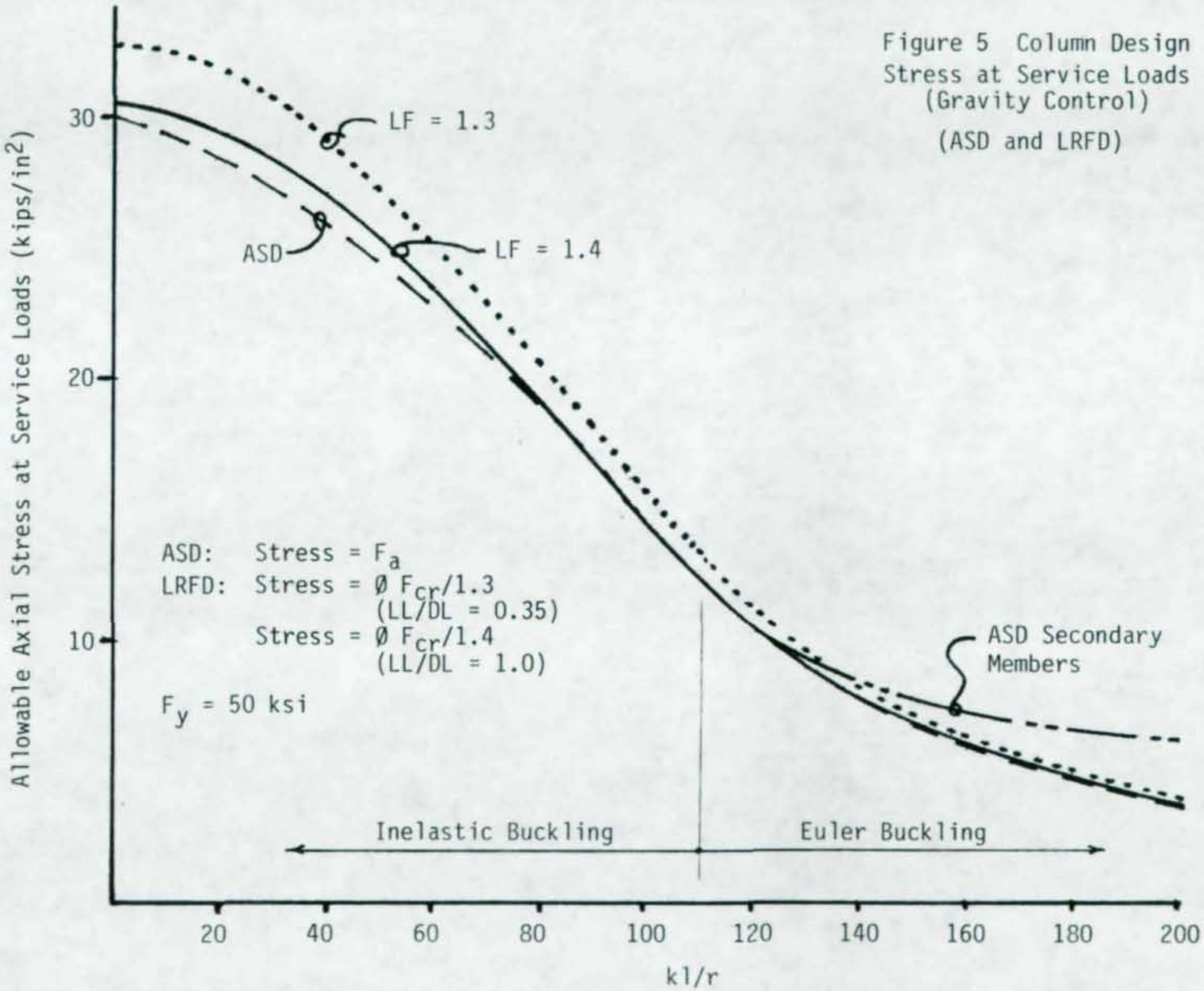


Figure 5 Column Design Stress at Service Loads (Gravity Control)

(ASD and LRFD)

columns supporting more than 2500 square feet of floor area) which gives a small LL/DL ratio (LF = 1.25 to 1.35), in combination with stocky columns (kl/r between 30 to 50) very nearly represents optimum column circumstances for savings with LRFD.

It is of interest to note that tension members have a different calibration level. In ASD, tension members controlled by yielding have the same capacity as a zero length compression member, i.e. 0.6 F_y. The calibration load factor for compression members is $\phi\ F_{cr}/0.6\ F_y = 1.42$ (kl/r = 0) and tension members is $0.9\ F_y/0.6\ F_y = 1.5$ (or LL/DL = 1.5, see Fig. 3). The same calibration for tension fracture is used since $0.75\ F_u/0.5\ F_u = 1.5$.

Comparing LRFD and ASD for wind-controlled column designs can be even more surprising. Figure 6 plots wind service load allowable stresses versus kl/r. Several facts should also be noted from this graph:

1. When the stress due to gravity load is approximately one half of the total applied axial stress, the LRFD LF can be approximately 1.1 (see Fig. 4). Then the LRFD allowable stress will be very similar to the ASD allowable stress with a 33% increase.

2. AISC ASD's 8th Edition permits the secondary or wind bracing member to have an increase in stresses at kl/r greater than 120. The LRFD committee did not find a rational justification for this (Ref. 15).

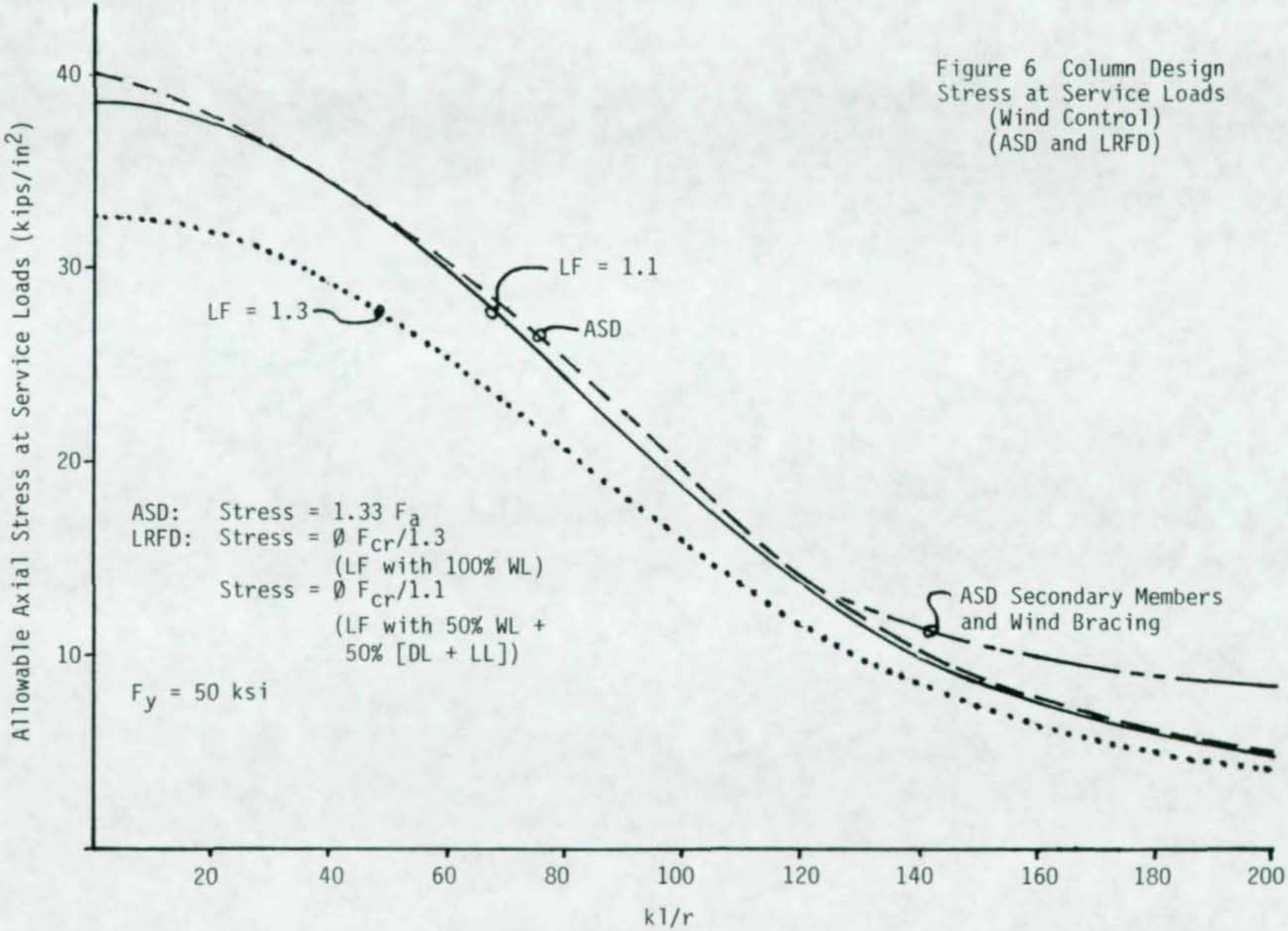


Figure 6 Column Design Stress at Service Loads (Wind Control) (ASD and LRFD)

3. A member with wind-generated axial stress and 0 or small amounts of gravity load will have a load factor at or close to 1.3 (see Fig. 4). This member will always be heavier in LRFD than ASD if controlled by stress and not deflection.

It can be observed, from figure 6, that an approximately 20% penalty on a LRFD LF = 1.3 wind brace is possible throughout the entire slenderness range. In fact at Kl/R = 200, the penalty can be greater than 50%.

Commentary on LRFD Column Design Procedures

The writer's review of column designs normal in the design office has shown that LRFD does permit significantly higher service loads than ASD for gravity-controlled situations and can permit significantly lower service loads than ASD for wind-controlled situations.

The LRFD column for LL/DL = 1.1, has a β of approximately 2.6 at the intermediate slenderness ranges (Ref. 7) although the LRFD was to be calibrated at a β of 3.0. The writer has noted that column LL/DL ratios can be significantly smaller in multi-story buildings which increases the probability that DL working stresses can reach the strength limit. (See Chapter IV Specification Review).

This small β occurred for the following reasons (Ref. 7):

1. Column strength variation has been shown by tests to be maximum at intermediate slenderness ratios.

2. Residual stresses and initial crookedness have their maximum effect at the intermediate slenderness ratios.

3. Related also to items 1 and 2; wide flange shapes in the larger sizes (flange thickness > 1-1/2") have a decreased resistance as compared to lighter column sections, against buckling in the intermediate slenderness range.

4. The LRFD curve is related to an initial crookedness of L/1500 (the mean) rather than L/1000 (the maximum) which gives a more uniform β.

This low β factor in the intermediate column slenderness range is very significant in the writer's opinion and has a major impact on the final factor of safety particularly when compared with the writer's concerns regarding load factors mentioned in Chapter VIII. AISC accepted this variance in the β factor because the present ASD equation has it also (Ref. 2).

The Guide to Stability Design Criteria for Metal Structures (Ref. 13) has proposed equations for multiple column curves (3 different curves depending on column type) and L/1000 initial crookedness. This would not significantly increase the complexity of the specification, and it would certainly increase its accuracy and the uniformity of the β index.

CHAPTER V

BEAM DESIGN BY LRFD

Specification Review

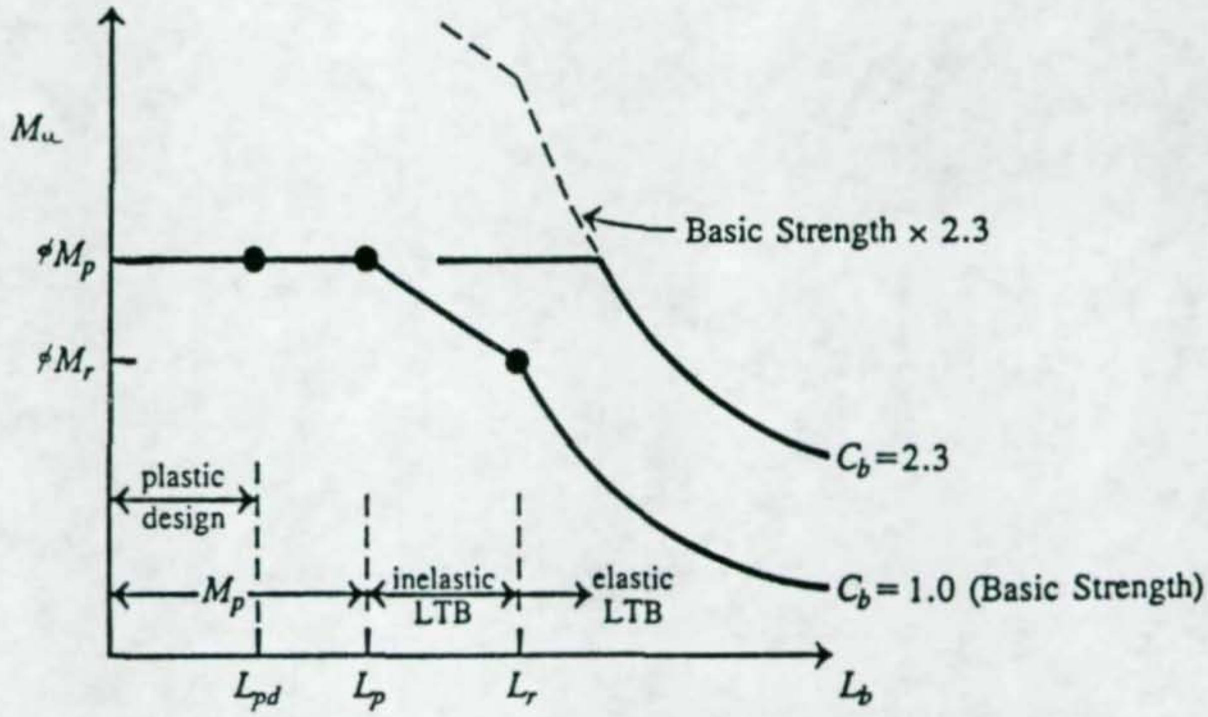
LRFD beam behavior has been divided into four separate regions based upon capacity and buckling characteristics (see Fig. 7):

1. The plastic region in which it is possible for the beam to attain plastic moment capacity and also sustain sufficient rotations to permit plastic analysis. $(0 < L_b < L_{pd})$

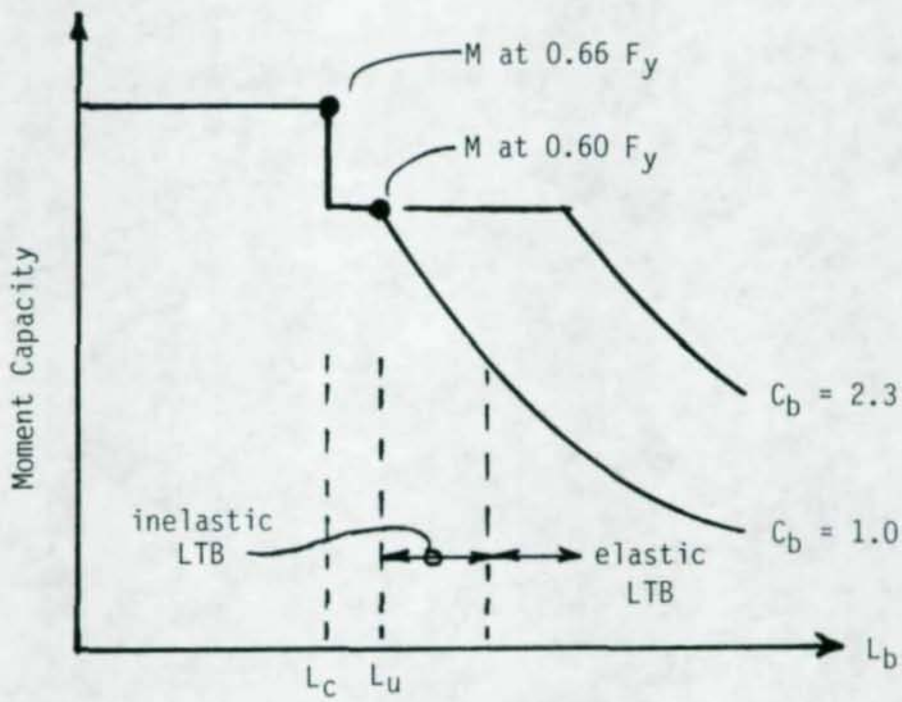
2. The area which is part of the inelastic noncompact buckling region where the beam can reach plastic moment capacity, but cannot undergo the rotations considered necessary to permit plastic analysis.
   $(L_{pd} < L_b < L_p)$

3. Inelastic buckling in the intermediate slenderness range where residual stresses and non-compact buckling control beam response.
   $(L_p < L_b < L_r)$

4. Elastic buckling is controlled by the flexural torsional buckling of the cross-section in an elastic state (i.e. buckling occurs before yielding). $(L_b > L_r)$



LRFD Moment Strength Curve (Ref. 2)



ASD Service Load Moment Capacity Curve

Figure 7 Moment Strength as a Function of Unbraced Length and Moment Gradient

As with ASD, all sections of the response curve are affected by moment gradient. The limit on plastic analysis criteria has moment gradient built into L_{pd} (eqn. F1-1) (Ref. 2) and C_b is a direct modifier of M_u for inelastic and elastic buckling. This C_b factor is the same as what is used in AISC ASD 8th Edition.

LRFD marks a significant change in flexural beam design. Features which may be noted from figure 7 include:

1. Integration of plastic analysis member selection with elastic analysis member selection. The plastic moment capacity is available for beams analyzed by plastic or elastic methods with the plastic analysis selection limited to the region where sufficient rotation capability is available.

2. The illogical discontinuity at L_c (allowable stress drops from 0.66 F_y to 0.60 F_y) does not occur in LRFD as in ASD.

3. The r_t value has been taken out of the main section of the LRFD specifications and the range of inelastic buckling is bounded by lateral stiffness of the cross-section (r_y) at the plastic moment range and the torsional properties of the cross-section at the interface with elastic buckling (flexural torsional buckling).

ASD used r_t (see definition in List of Symbols) as a measure of the torsional buckling strength of the member's cross-section. The writer believes the lateral torsional buckling failure criteria will now be more easily understood particularly by students.

4. Residual stresses within the beam have been considered in the inelastic buckling region similar to the column design approach.

Shear Design for normal wide flange beams has not changed significantly except that it is calibrated at a load factor of 1.35 (LL/DL = 0.65). This can be shown as follows:

$\phi = 0.9$

$V_u = 0.6\ F_y\ A_w$ (LRFD)

$V = 0.4\ F_y\ d\ t$ (ASD)

$$\text{Calibration LF} = \frac{0.9 \times 0.6}{0.4} = 1.35$$

Beam Flexural Design Compared

The specification response curves for four beams were plotted comparing the LRFD and ASD moment capacity at service levels versus unbraced length. These beams were selected to have similar moment capacities when braced, but they have varying slenderness parameters (i.e. the compression flange width (b) varies).

The following can be observed from figures 8 through 11.

1. The LRFD curves are much smoother, implying, a better representation of beam behavior.
2. L_c (ASD) is approximately the same as L_p (LRFD).
3. Beams which have cross-sections considered as stocky beams (i.e. midway between a slender beam member and a column member) with a resulting shape factor of about 1.1, have been calibrated at a LF = 1.5 (LL/DL = 3.0). This calibration occurs out to the Lu (ASD) point beyond which LRFD gives substantially larger moment capacities in the elastic and inelastic buckling region.
4. Slender cross section beams typically have a higher shape factor and therefore, it can be seen their capacities will be proportionaly higher than stockier beams. It can be shown that the proportional increase is: $\left(\frac{Z/S - 1.0}{1.1}\right) \times 100\%$
5. Allowable LRFD moment capacities beyond Lu are significantly larger than ASD. At the calibrated (LL/DL= 3) ratio; unbraced lengths beyond Lu can have 20% to 40% higher moment capacities with LRFD than ASD. At LF = 1.4 (LL/DL ratio = 1.0) moment capacities can be 30% to 50% higher for the LRFD procedures.

Commentary on LRFD Beam Design Procedures

LRFD has taken flexural design of beams to a new milestone. Significantly more capacity is available in the elastic flexural-torsional buckling region with resulting economies.

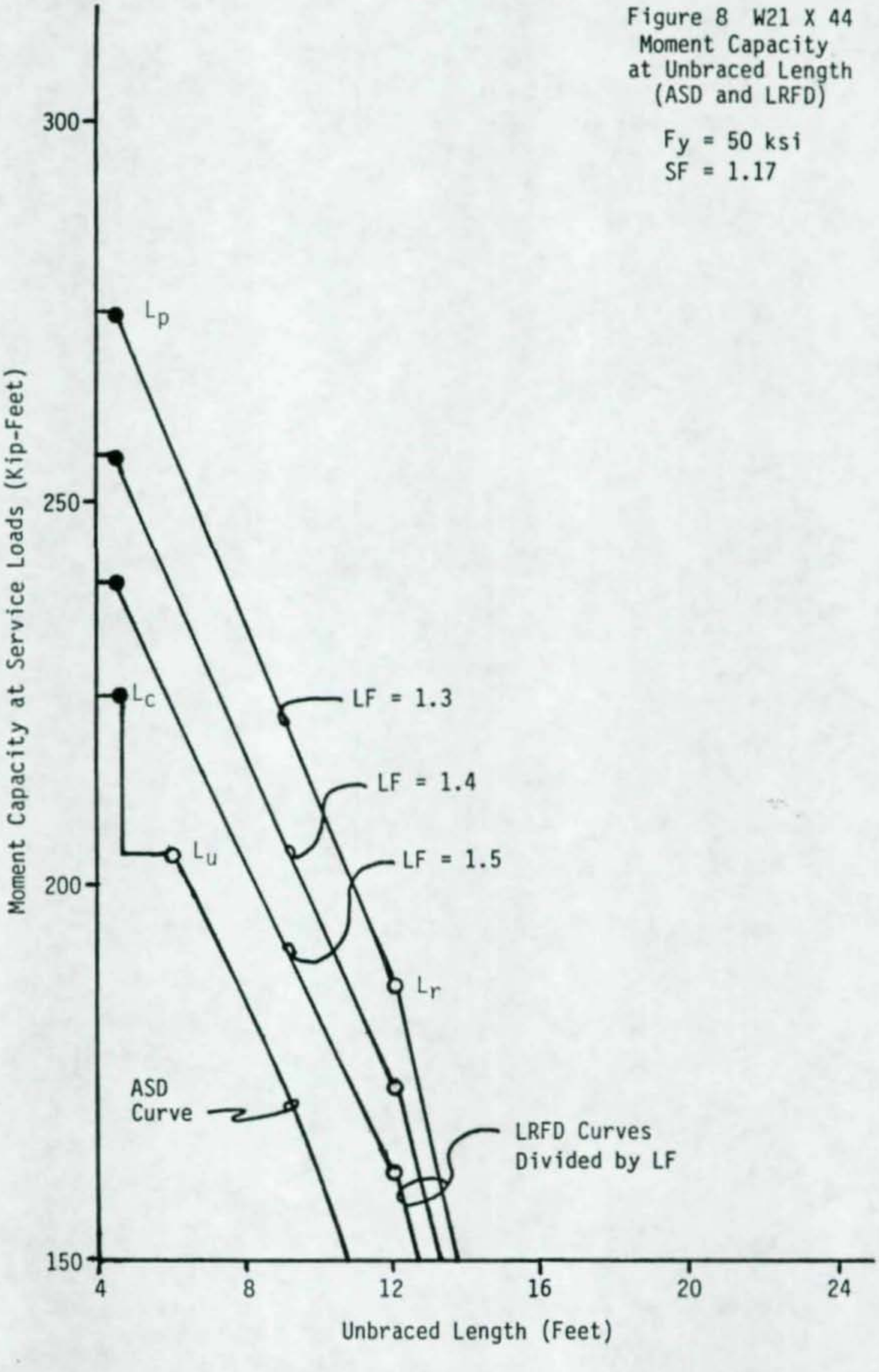
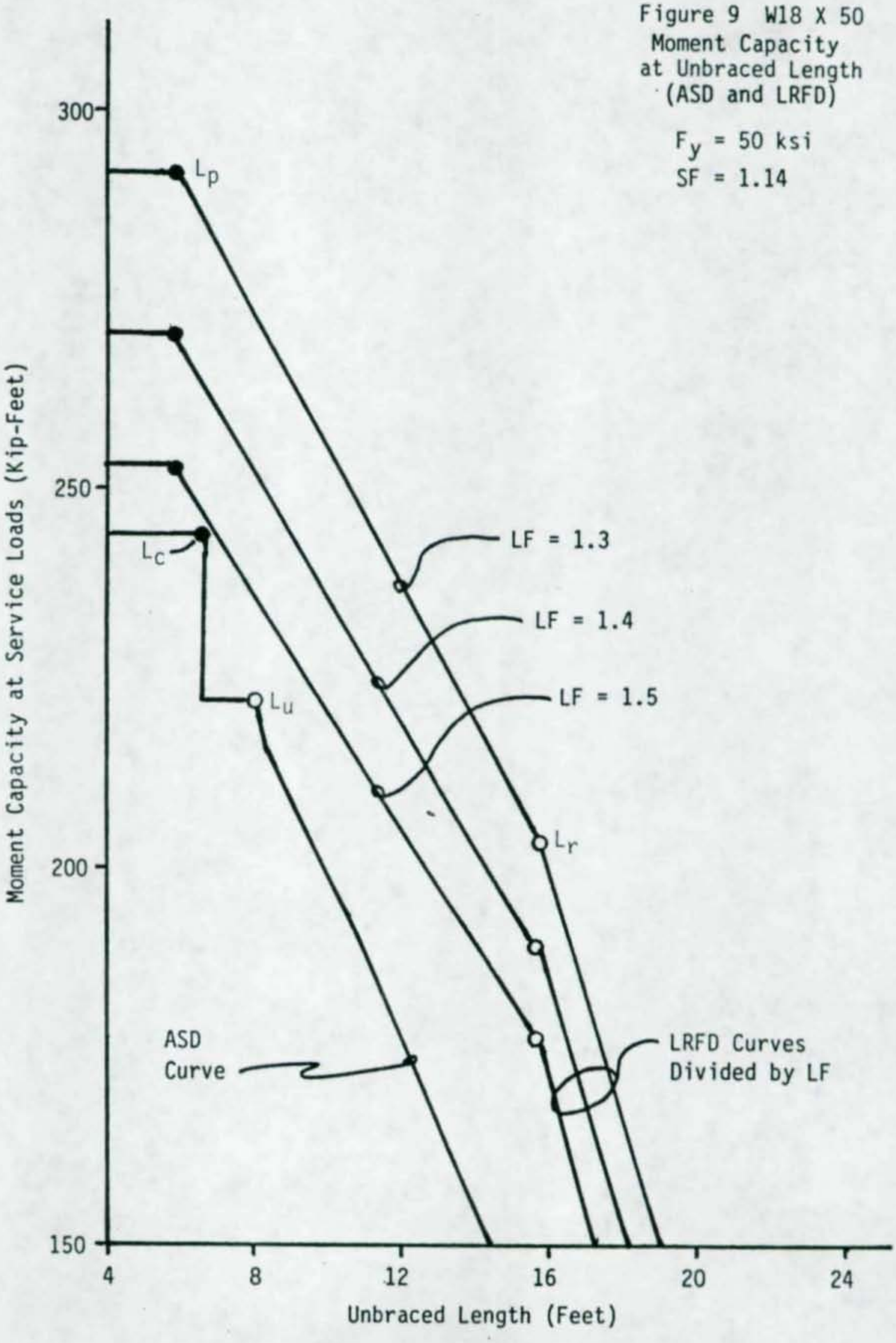


Figure 8 W21 X 44
Moment Capacity
at Unbraced Length
(ASD and LRFD)
Fy = 50 ksi
SF = 1.17
Moment Capacity at Service Loads (Kip-Feet)
300
250
200
150
Lp
Lc
Lu
Lr
LF = 1.3
LF = 1.4
LF = 1.5
ASD
Curve
LRFD Curves
Divided by LF
4
8
12
16
20
24
Unbraced Length (Feet)

Figure 9 W18 X 50
Moment Capacity
at Unbraced Length
(ASD and LRFD)

F_y = 50 ksi
SF = 1.14



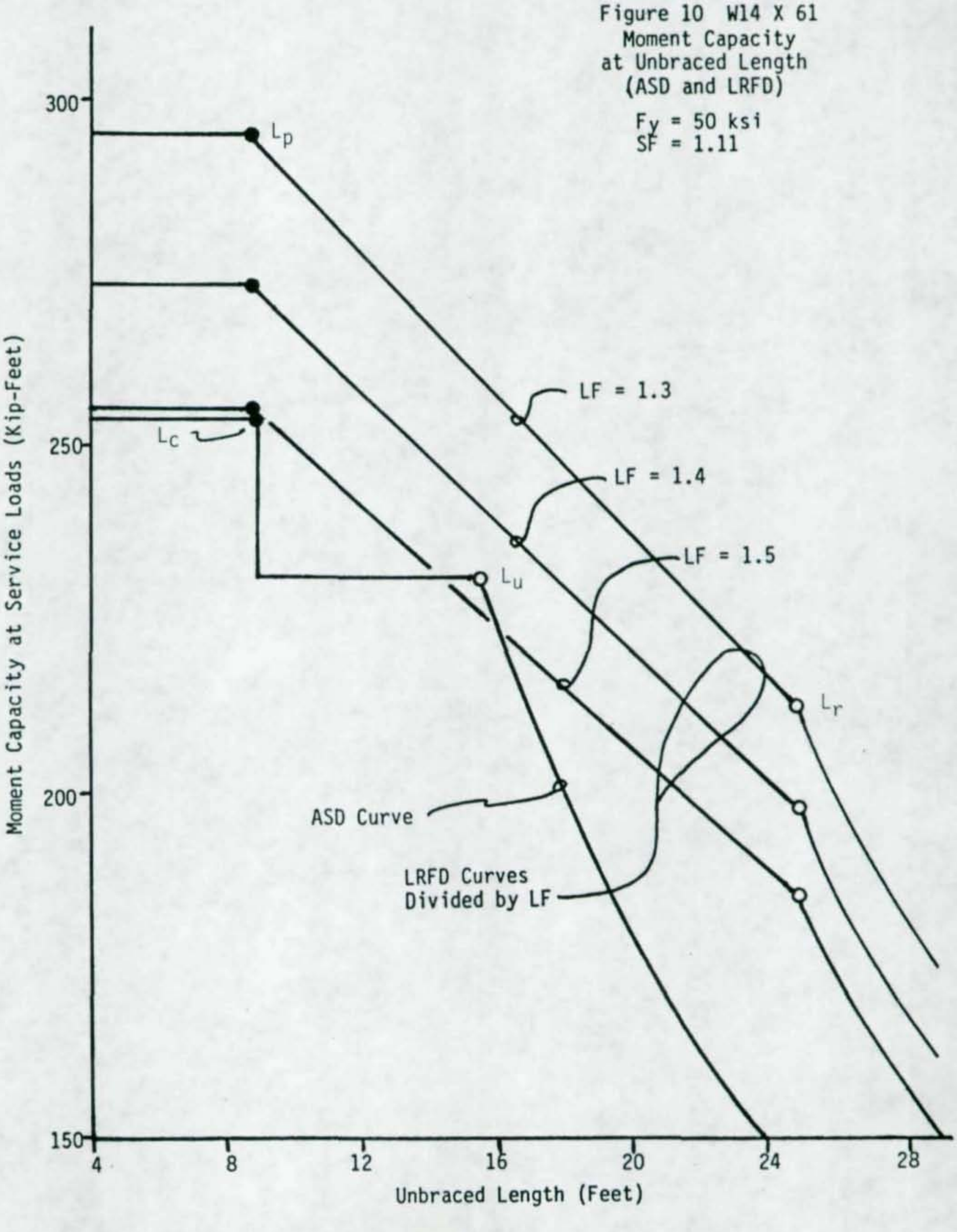


Figure 10 W14 X 61
Moment Capacity
at Unbraced Length
(ASD and LRFD)
Fy = 50 ksi
SF = 1.11
Lp
Lc
Lu
Lr
LF = 1.3
LF = 1.4
LF = 1.5
ASD Curve
LRFD Curves
Divided by LF
300
250
200
150
4
8
12
16
20
24
28
Moment Capacity at Service Loads (Kip-Feet)
Unbraced Length (Feet)

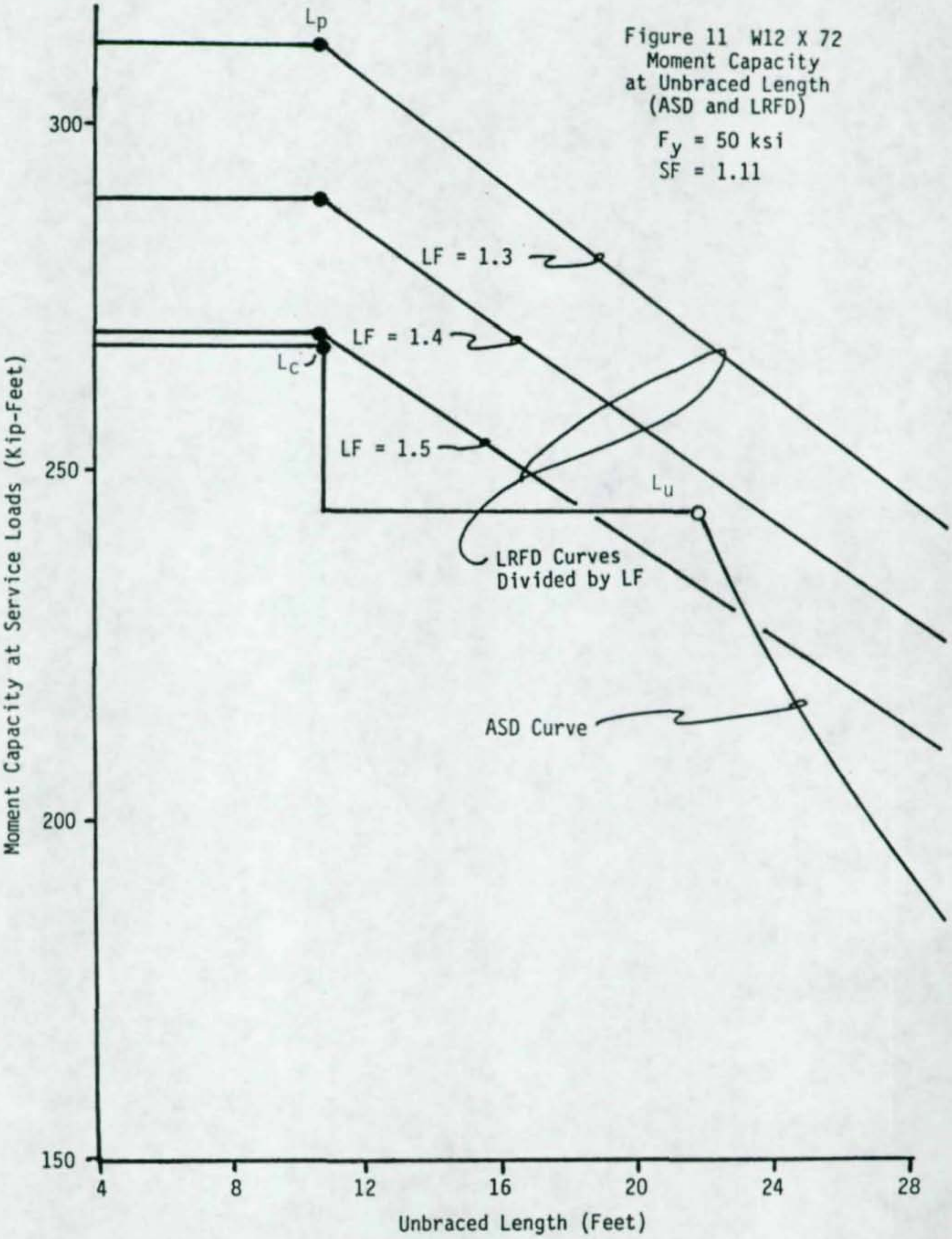


Figure 11 W12 X 72
Moment Capacity
at Unbraced Length
(ASD and LRFD)
F_y = 50 ksi
SF = 1.11
L_p
L_c
L_u
LF = 1.3
LF = 1.4
LF = 1.5
LRFD Curves
Divided by LF
ASD Curve
300
250
200
150
4
8
12
16
20
24
28
Moment Capacity at Service Loads (Kip-Feet)
Unbraced Length (Feet)

Assuming that the LL/DL (ratio = 3) calibration is the correct index to aim for, the profession does not yet agree upon the correct ϕ. AISC LRFD ϕ for flexure is 0.9. The original formulation was for a ϕ = 0.86 (Ref. 28) and recent information is proposing a variable ϕ from 0.81 to 0.90 for flexural members to maintain a uniform β = 3.0 (Ref. 25).

It is not surprising that the ϕ for shear (AISC LRFD ϕ = 0.9) is considered too low for β = 3.0. (Ref. 25). This ϕ may have been chosen because of the disastrous consequences of a shear yielding failure.

CHAPTER VI

COMPOSITE BEAM DESIGN BY LRFD

Specification Review

AISC has adopted the strength design approach for LRFD composite beam design similar to steel beam plastic design or concrete beam flexural strength design.

ASD procedures are based upon a transformed composite beam section and stresses are limited in the concrete (compressive) and steel bottom flange (tensile) to permissible allowable stresses of 0.45 f'_c and 0.66 F_y respectively (see Fig. 12). Partial composite action is addressed with a convex parabolic function from full composite action capacity to the capacity of the non-composite steel beam alone, based upon the quantity of shear studs provided.

LRFD utilizes the entire cross-section of the steel beam at yield in tension or in a combination of tension and compression. Only the top of the concrete slab is used as a compression block. See figure 12. Partial composite action is also based, as in ASD, on the quantity of studs furnished. The quantity of studs establishes the amount of compressive force in the concrete flange which can be transferred into the composite steel section, and thereby locates the neutral axis. Varying the amount of studs moves the neutral axis up and down

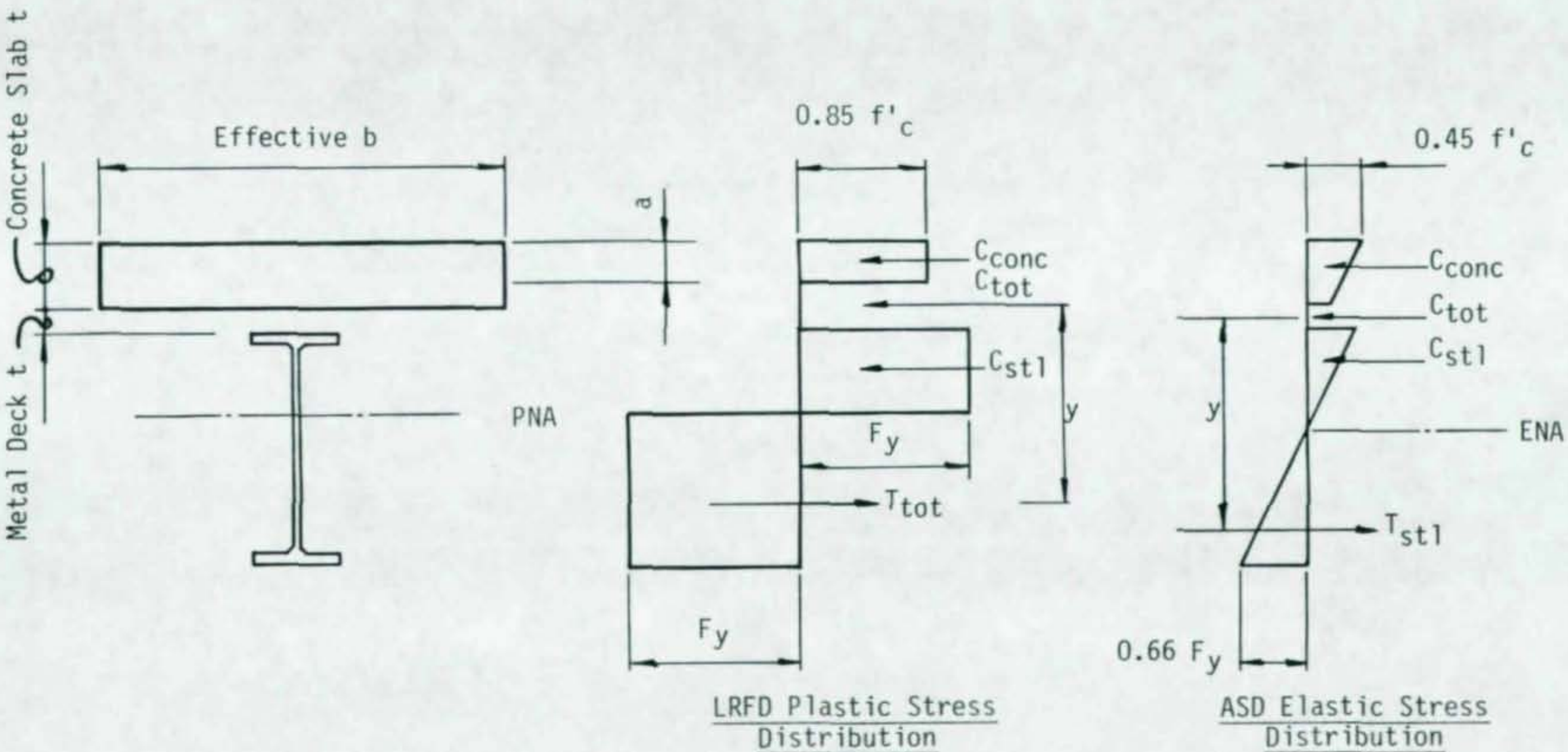


Figure 12 Composite Beam Philosophies
(LRFD and ASD)

the depth of the composite beam and varies the beam's strength capacity. The effective quantity of studs can not exceed that required to develop the strength of the concrete flange in compression ($0.85\ f'_c\ A_c$) or the steel beam in tension ($A_s\ F_y$).

As design aids, the LRFD manual tabulates moment strength capacities for seven different neutral axis locations from the top of the steel beam's top flange to a location on the steel beams web which gives the minimum code allowable stud quantity ($Qn = 0.25\ A_S\ F_y$). The ordinate on this chart is the distance from the top of the steel beam's top flange to the center of the concrete compressive force. This will vary with percent composite action, concrete flange width and/or strength, and concrete flange geometric properties such as thickness and direction of the steel deck ribs.

The LRFD tables require an iterative solution to obtain a final design solution optimizing the amount of studs required. Design charts have also been published which simplify selection of the most economical beam (Ref. 27).

Composite Beam Flexural Design Comparison

A selection of composite beams were designed by the writer for both LRFD and ASD criteria for two office design projects. The spans ranged from 25' to 40' and live loads varied from 70 to 100 psf. The average LRFD savings was 13% of the fabricated steel product cost including studs. (See Chapter VII).

Zahn (Ref. 29) has done an exhaustive study of ASD vs. LRFD designed components. His final report included this analysis:

1. Strength - For a conservative study (high LL/DL), with a 250 psf live load (LL/DL = 4.2, LF = 1.52), LRFD showed consistent savings of 10% to 15% (by weight) from the ASD member at all span lengths from 10' to 45'.

2. Strength and Vibration Control - The second major study was for a more normal live load of 100 psf (LL/DL = 2.0, LF = 1.4) and was also subjected to Murray's vibration check (Ref. 21). The conclusion of this study was that vibration was only a problem at spans less than about 23 feet and above this range the strength criteria of LRFD again saved from 10% to 20% from the ASD member weight.

   Note: Vibration serviceability checks for steel beam framed floor systems are usually based upon the Murray method. The stiffness and frequency of the floor system is compared to the amount of damping estimated to be present from the structural framing, ceiling, partitions and mechanical systems.

   The writer does not agree with Zahn in his using full live load on the system for calculating the beam frequency. Vibration problems usually occur with minimal transient load since the more mass is on the system, the more damping is likely to be available and the smaller the frequency will be.

3. Deflection - Transformed moment of inertia values from LRFD procedures can be significantly less than those from ASD. AISC has reduced these values because ASD values were too unconservative.

4. Shear stud requirements - LRFD composite beam design typically reduces the amount of shear studs required in the range of 10% to 30%. This also depends upon the beam chosen.

The conclusion of Zahn's article was the LRFD composite design had average cost savings of 14% to 15% on spans from 18 to 45 feet. Spans less than 18 feet saved approximately 6%.

Commentary of LRFD Composite Beam Design Procedures

ASD composite beam design procedures are a mixture of elastic stress distribution calculation procedures with limit state and yielding assumptions. The limit state assumptions referred to include:

1. Calculation of total load elastic stresses on transformed section ignoring superposition of dead load stresses in noncomposite unshored beams.

2. The use of the convex parabolic strength interaction as opposed to a straight line equation for partial composite action is rational only on the basis of test results.

LRFD carries the composite design procedure to its next logical step by designing in a similar fashion to plastic design of steel beams and ultimate strength design of concrete beams. Comments the writer would like to make include:

1. The use of the compression block (a x b) as the effective concrete area in calculating transformed section properties and shear stud quantities is much more rational than ASD provisions. A composite girder design has often had less ASD structural capacity when used with a concrete slab on metal deck than the corresponding composite beam design. This anomaly occurs because the A_c will be larger with the girder since it is parallel with the deck flutes which adversely affects neutral axis position and increases shear stud count with a concrete control situation.

2. Vibration did not control any of the composite beam selections in the case studies using the Murray method (Ref. 21). Vibration is a real problem, however, for steel

structures (and some concrete structures) and any "lightening" of component flexural members must be made cautiously.

3. Strength design of composite beams will require the shear studs to be more highly stressed under working loads. The coefficient of strength variation of shear studs welded through metal deck can be very large. The writer has experienced a 50% failure rate on one of his jobs, with a 10% to 30% failure rate not unusual.

CHAPTER VII

BUILDING DESIGN CASE STUDIES: LRFD VS. ASD

Introduction

The major framing components from three structures were analyzed and designed by both LRFD and ASD methods. A design office comparison of the two specifications would be difficult to make without actually examining "real world" examples. This is particularly important because of the significant impact of varying load factors with respect to load ratios. These structures were selected for the following reasons:

1. The three structures in the design study encompass a fairly broad range of different type of components and loading that are typical of normal design office work.

2. All three structures were already designed in steel using ASD procedures and were under construction during the preparation of this thesis.

3. The writer did the original ASD work and was, therefore, cognizant of the difference in complexity and design effort required between ASD and LRFD.

Each case study's design has been summarized in tabular form. The LRFD and ASD designs are summarized and compared on the basis of structural quantities, cost, and savings.

Cost comparisons are based upon fabricated structural steel cost with the differential for high strength steel (A572-50) an additional amount as is the cost of cambering and installed shear studs where required.

It was not necessary to include erection costs because that is directly proportional to the piece count and would be identical for both designs. (All unit prices current as of November 1988, Ref. 26).

Arena Complex

Project Description

This complex is in Cincinnati, Ohio and is a combination arena for basketball and volleyball, racquetball courts, and support facilities for the athletic department.

The grade and below grade structure is of cast-in-place concrete flat slab construction, supporting as much as 250 psf superimposed loading from the arena floor above. The arena alone has approximately 90,000 square feet of gross area and will seat 13,000 spectators.

The arena roof and wall construction is framed with structural steel members. The one-way 20' deep roof trusses span 239' to columns 36' on center. Standard roof joists span the 36' between the deep trusses and span 78' between the deep trusses and the walls. See figure 13 for a plan view of the arena.

Stability is provided by vertical diagonally braced wind bents in the east-west direction and unbraced rigid frame action in the north-south direction.

The arena roof loading consists of:

Live Loads

Snow Load: 25 psf (no obstructions to cause drifting).
Mechanical Load: 5 psf *
Catwalk Load: 3 psf *
Future Ceiling Load: 5 psf*

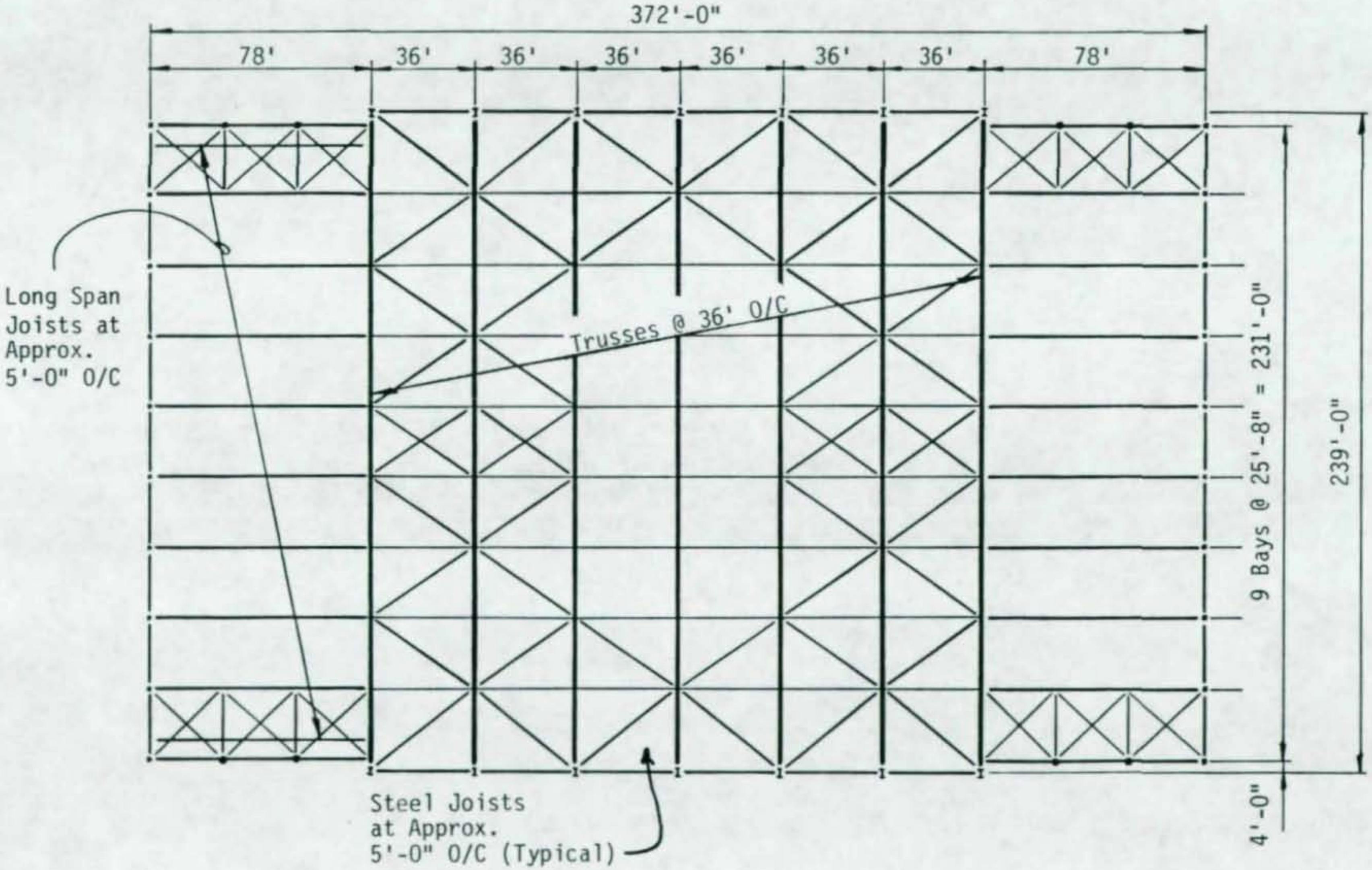


Figure 13 Arena Roof Plan

Dead Loads

Truss Selfweight:	27 psf
Roof deck and joists:	5 psf
Roofing and ballast:	17 psf
Total	87 psf LL + DL

*Considered as live load for LRFD

Additional loading from four - 47,000 lb. each mechanical units within the truss depth was considered as dead load.

Design Comparison

A design comparison for the deep roof trusses, the supporting beam-columns, and the major bracing was done with ASD and LRFD. Table 1 gives a complete breakdown of tonnage and cost comparisons.

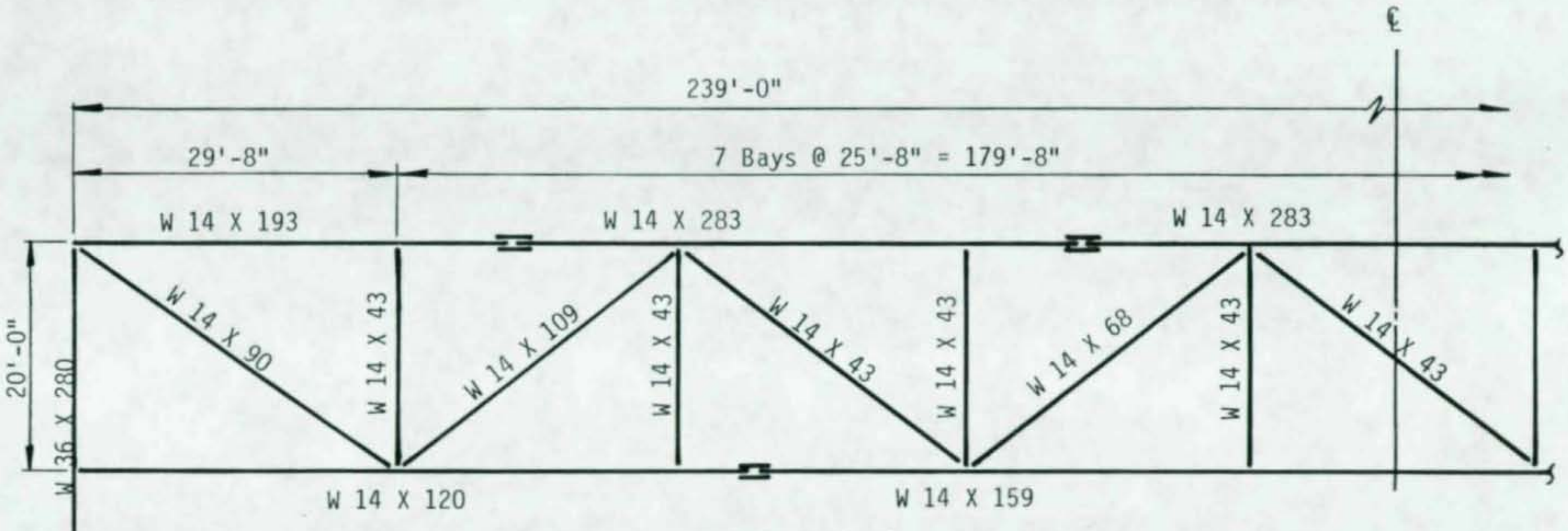
Figure 14 details the component changes between the two design methodologies for the typical truss. A fact to consider is that wind forces are negligible in the trusses themselves, in that gravity controls the member sizes. As expected, LRFD saves about 14% of the tonnage. This savings is in both tension and compression members and is due to the following:

1. Average load factor of 1.31 for live loads and dead loads. As noted previously in column design, compression members are calibrated at a LL/DL ratio of 1.1 (average load factor of 1.41) and tension members have been calibrated similar to other component designs at a LL/DL ratio of 3.0 (average load factor of 1.5).

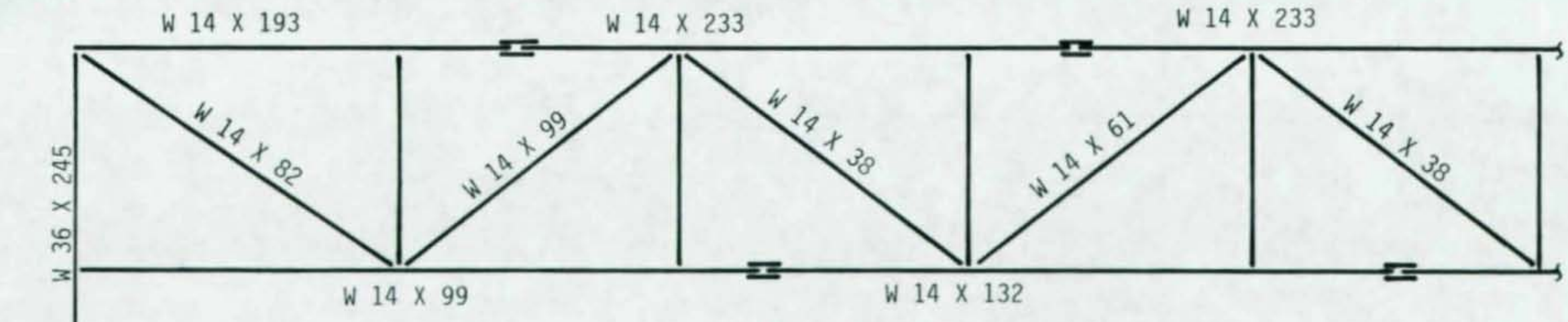
Table 1 Comparison of Arena Structural Components
Designed by ASD and LRFD

	ASD	LRFD
Trusses		
Tonnage Weight (F_y)	58T (50)	50T (50)
Fabricated Cost	$38,100	$32,850
Savings	-	14%
Columns		
Size (F_y)	W36 x 280 (50)	W36 x 245 (50)*
Fabricated Cost	$7360	$6440
Savings	-	12.5%
Wind Bracing		
Size (F_y)	W10 x 49 (36)	W12 x 65 (36)
Fabricated Cost	$665	$880
Savings	24.5%	-

* This design satisfies the strength requirement, but its use would increase lateral drift approximately 15% which may be inacceptable.



ASD Truss Design



LRFD Truss Design

Figure 14 Arena Roof Truss Design Comparison
(All Steel F_y = 50)

Note: ANSI load combinations successively applies 1.6 and 0.5 load factors on snow and live load when in combination. The writer believes it would be unconservative to use less than a 1.2 load factor (same as dead) on live loads in this combination for this structure. A strict ANSI interpretation would have an average load factor of 1.23 on this truss design with even greater savings in member sizes.

2. Efficiency of beam-column design. The major cost savings are in the top and bottom truss chords. The wide flange chords are on their sides with weak-axis bending from loads applied between panel points and secondary truss moments. LRFD's use of the plastic section modulus (shape factor of 1.5 for weak axis bending) provides a benefit in strength in addition to the savings from the low load factor.

The columns supporting the trusses were also compartively designed by ASD and LRFD. The 36" wide flange columns not only support the trusses, but they provide stability and wind resistance through rigid frame action with the trusses. The writer had initially assumed that these columns would have a larger required size with LRFD since the wind moment is more than twice the live load moment. This was not the case for the following reasons:

1. Average load factors were 1.0 and 1.12, axial and bending respectively. Note the writer again used a load factor of 1.2 minimum on live load rather than the 0.5 permitted by ANSI with load combinations with wind. These load factors (1.0 and 1.12) are close to the ASD/LRFD wind synchronization for column design as discussed in Chapter IV (see Fig. 6).

2. The more liberal beam strength equation again consistently gives larger effective capacities than ASD at unsupported lengths exceeding L_p. (See figure 9 for example).

The east-west bracing was the other major component compared. The total axial stress in the compression strut is due to wind, giving a load factor of 1.3. The ASD selected component is a W10 x 49 with a kl/R of 200. As noted previously in Chapter IV (see Fig. 6), the AISC ASD permits about 50% more effective strength at this slenderness in a wind control situation. That was the case here, with a LRFD column brace required to be a W12 x 65.

In summary, LRFD component sizes are about 13% lighter in tonnage than the comparable ASD components for this arena project. The major reason is the low live load to dead load ratio which has a load factor about 13% lower than the AISC calibration load factor. The other interesting point to consider is the ASD compression wind brace being 100% overstressed in an LRFD design check.

Low Rise Office Building

Project Description

This four story building is in northern Hamilton County, Ohio and is a combination owner/tenant occupied building. The building had to be economical and flexible for tenant requirements, but the owner wanted more than the code-minimum 50 psf live load for his own usage requirements. An electrified floor fill system was installed on top of the structural slab to allow for frequent tenant layout changes.

This 125,000 square feet gross area structure has 25' x 30' bays. As is usual in grid frameworks, the most economical and shallowest grid has the beams spanning the long direction and the girders spanning the short direction. See figure 15. The girders were recessed 1-1/2" into the concrete floor slab to create additional headroom below.

Floor construction consists of composite steel beams and girders with a 5-1/2" structural slab (2" composite deck plus 3-1/2" regular weight concrete) (Fig. 16). The 2-1/2" electrified topping slab is a delayed pour to allow for tolerance in levelness of the structural system. Roof construction is a grid of steel beams and girders supporting metal roof deck.

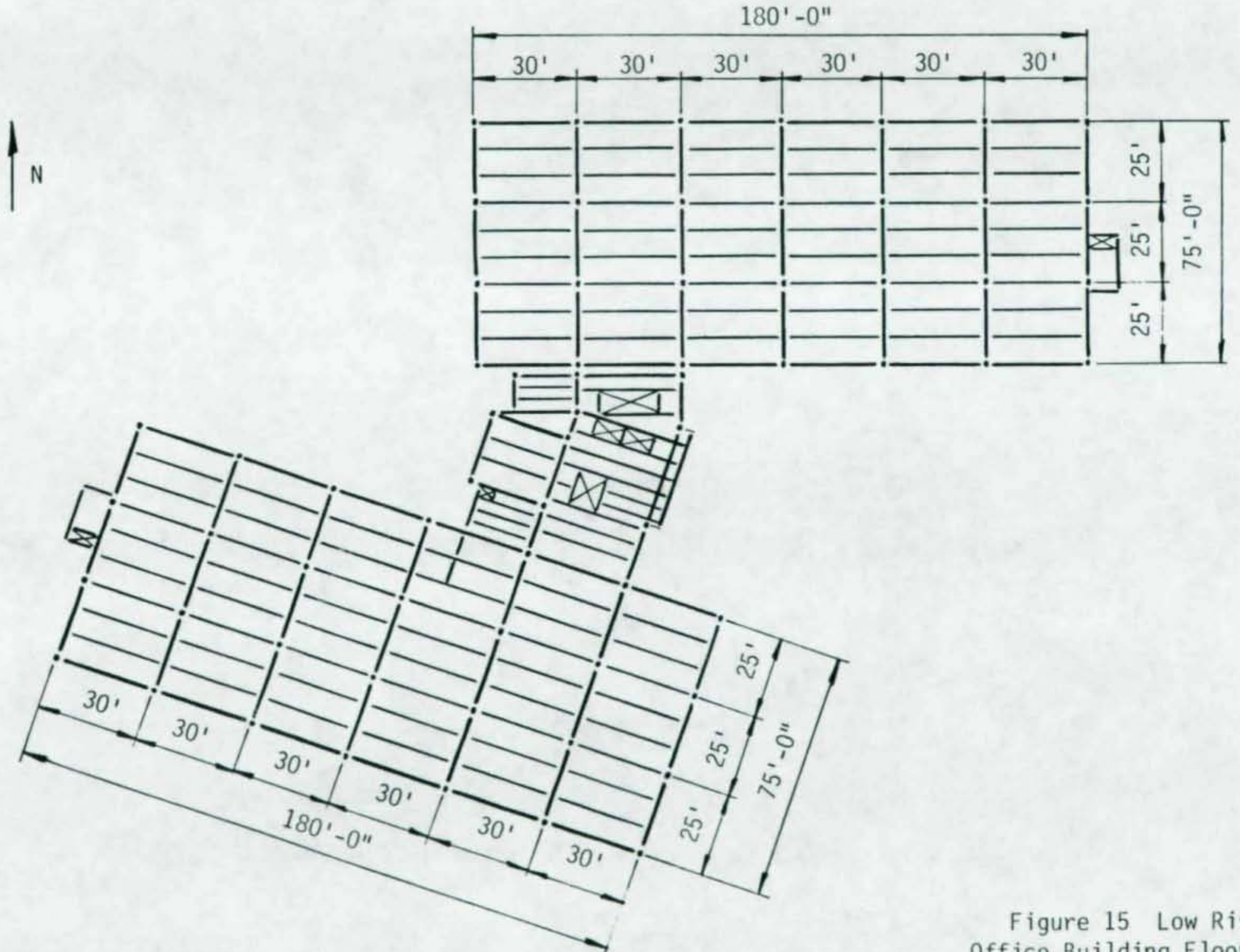


Figure 15 Low Rise
Office Building Floor Plan

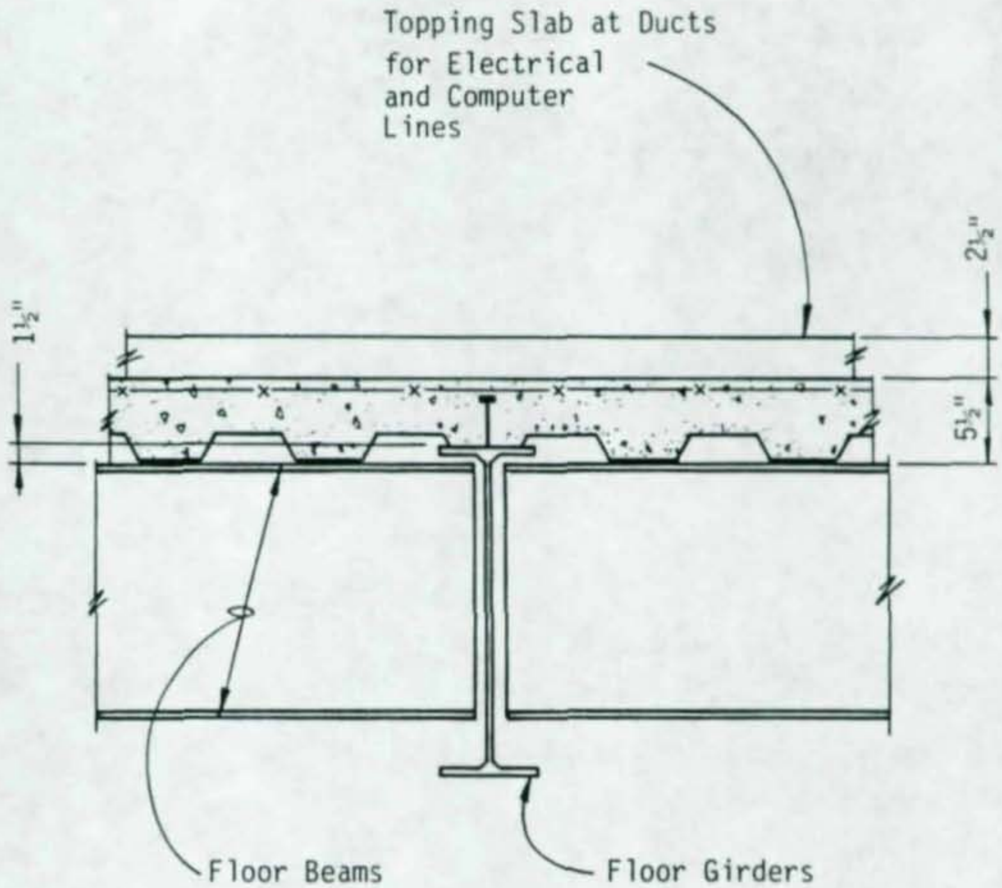


Figure 16 Composite Beam/Girder Section

Lateral loads are resisted by unbraced moment resisting frames in both directions. As in all three structures in this study, wind controlled the lateral load design (as opposed to earthquake forces).

In addition to saving steel tonnage, another advantage of the girders spanning the 25' direction was in resisting lateral loads. Wind loads are maximum in the transverse direction and the girders are best able to control the stresses and deformations. The girders were "pinned" at the exterior column and were designed continuous at the two strong-direction interior columns for wind loads (see Figs. 15 and 17) to form 3 girder/2 column transverse wind bents.

The perimeter beam/spandrels and the two exterior longitudinal lines of columns form the wind bents in the opposite direction. In this manner, strong direction framing provides stability in both directions without resorting to weak-axis bending in the columns. This also avoids the under-designed end column problem which developes because of unbalanced gravity moment and is typical of semi-rigid construction (Ref. 20).

All steel to steel connections are partially restrained connections. All connections of beams to girders and beams or girders to weak-axis columns are "pinned" PR connections, and all beam or girder connections to strong-axis columns are "semi-rigid"/"wind clip" PR connections.

"Semi-rigid" PR connections have historically been designed to yield under gravity loads, "shake-down" and form a hysterisis loop with lateral loading. The writer has designed several buildings to take advantage of this hysterisis loop and composite action by adding reinforcing within the slab depth to increase the moment arm, damping from the concrete and girder stiffness.

The floor loading consists of:

Live Loads:		
Beams:	100 psf *	
Girders and columns:	70 psf *	
Dead Loads:		
Electrified fill slab:	35 psf	
Concrete structural slab:	65 psf	
Ceiling/mechanical:	5 psf	
Total	205 psf	(unreduced)
	175 psf	(reduced)
Facade brick and light gauge metal framing	60 psf	

* Includes 20 psf partition load which ANSI A58.1 considers as live load.

Design Comparison

The design of floor beams and girders, roof framing and columns was accomplished by ASD and LRFD. Table 2 details the breakdown in resulting component sizes.

Pertinent information for comparing the results includes:

a. The floor framing members have a 7% to 20% cost savings with LRFD. The typical bay framing comparison is summarized in Fig. 17.

Table 2 Comparison of the Low Rise Office Building Components Designed by ASD and LRFD

	ASD	LRFD
Beams		
Size (F_y)	W16 x 31 (50)	W16 x 26 (50)
Studs	22	20
Fabricated Cost	\$338.50	\$286.25
Savings	-	15%
Girders		
Size (F_y)	W21 x 50 (50)	W21 x 44 (50)
Studs	60	26
Fabricated Cost	\$500.50	\$400.25
Savings	-	20%
Perimeter Spandrel		
Size (F_y)	W21 x 44 (50)	W21 x 44 (36)
Studs	10	12
Fabricated Cost	\$376.25	\$348.50
Savings	-	7%
Beams (Mechanical Room, LL = 175 psf)		
Size (F_y)	W18 x 40 (50)	W18 x 35 (50)
Studs	28	27
Fabricated Cost	\$436.25	\$385.50
Savings	-	12%
Girders (Mechanical Room, LL = 140 psf)		
Size (F_y)	W21 x 68 (50)	W21 x 57 (50)
Studs	80	60
Fabricated Cost	\$678.50	\$558.00
Savings	-	18%
Roof Beams		
Size (F_y)	W12 x 14 (50)	W12 x 14 (36)
Fabricated Cost	\$138.00	\$126.00
Savings	-	8.5%
Roof Girders		
Size (F_y)	W16 x 26 (50)	W16 x 26 (50)
Fabricated Cost	\$213.50	\$213.50
Savings	-	-

(Table 2 continued)

	ASD	LRFD
Interior Columns Size (F_y) Fabricated Cost Savings	 W10 x 68 (50)* $1162 -	 W10 x 68 (50)* $1162 -
Exterior Columns Size (F_y) Fabricated Cost Savings	 W10 x 54 (36) $843.75 21%	 W10 x 68 (36) $1062.50 -

* Column sizes controlled by lateral drift; ASD and LRFD column, W10 x 60 (50) and W10 x 54 (50) respectively for strength

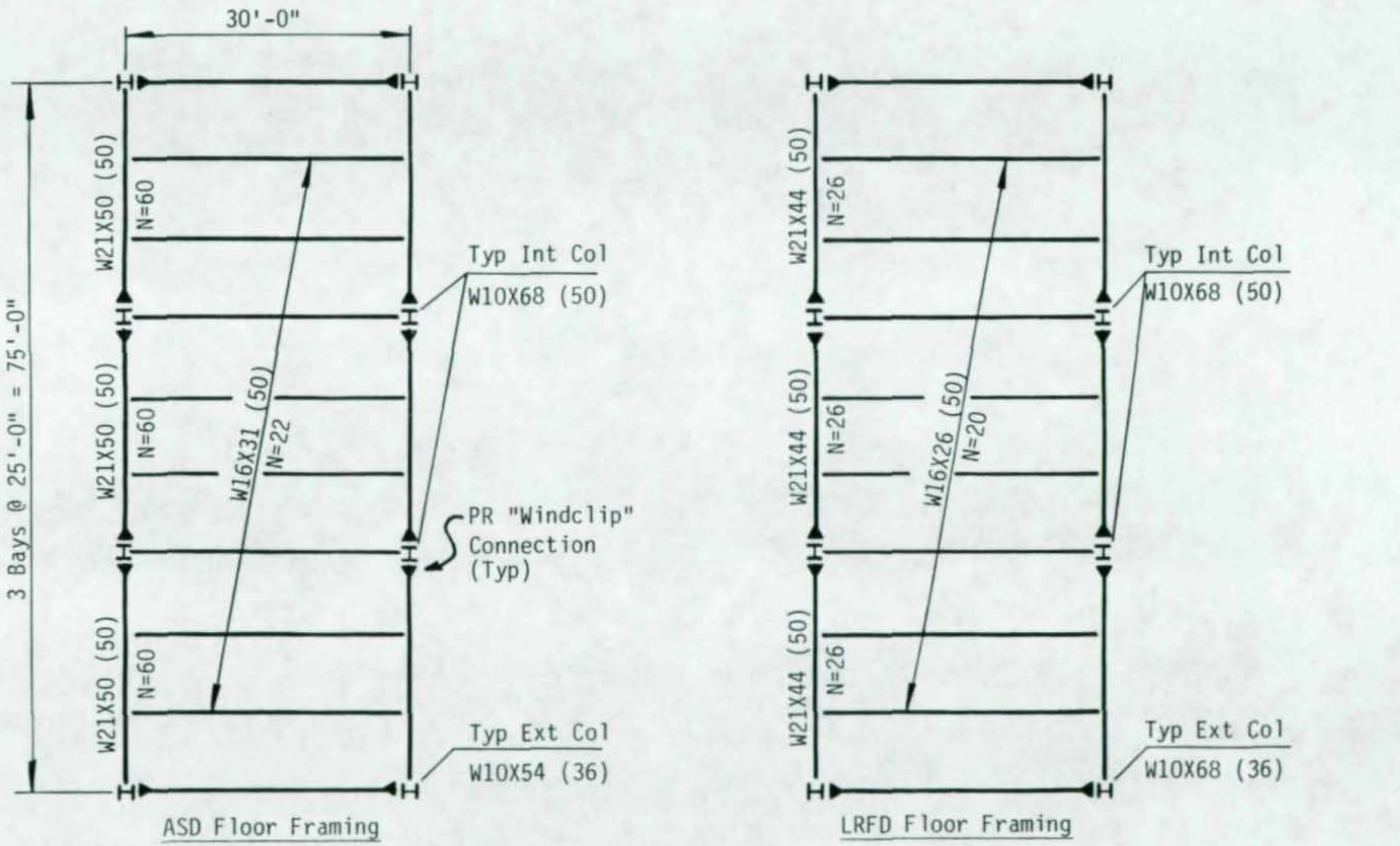


Figure 17 Low Rise Office Building
Floor Framing Design Comparison

b. Load factors vary from 1.35 to 1.45 for the design of floor members with the 1.45 load factor at the mechanical room.

c. Stiffness controlled the design of the perimeter spandrel (LRFD savings of 7%) for vertical deflection and the interior column (identical LRFD and ASD members) for lateral story drift.

d. Exterior columns exhibited a 21% cost savings with ASD because of the substantial wind generated moment and the low LL/DL ratio which decreases the effect of the wind load combination equation (see Fig. 4).

Vibration is often a problem in composite framing particularly in this span range based upon the writer's experience. The LRFD floor beams and girders were checked for vibration potential with the Murray frequency/damping equation (Ref. 21 and Chapter VI) as were the original ASD members. Vibration did not control any minimum sizes, however, because of the additional mass of the topping slab and the resulting decrease in frequency of the system.

High Rise Office Building

Project Description

This twenty-one story building in Dayton, Ohio is being developed, built and owned by the same entity. The 450,000 square feet gross area granite-clad tower/lowrise complex will be in the center of Dayton's planned entertainment block.

Two levels of underground garage/mechanical areas are framed in cast-in-place concrete flat slab construction. The steel columns for the tower continue down through these basement levels to the foundations and are encapsulated in the concrete columns supporting the below grade construction.

Typical floor construction consists of composite W21 beams spanning 40' with W21 and W30 girders spanning 30' and 35' respectively. The slab is 6-1/4" thick (3" composite deck plus 3-1/4" lightweight concrete) to maintain a two hour non-protected slab fire rating. The granite-clad precast panels are supported near the ends of the perimeter spandrel beam/girders.

This height of building is outside what is considered the economical range of moment resisting frames for resisting lateral loads. A space frame braced core (see Fig. 18) was designed for 100% of the lateral load allowing the remainder of the columns to be designed for only gravity loads.

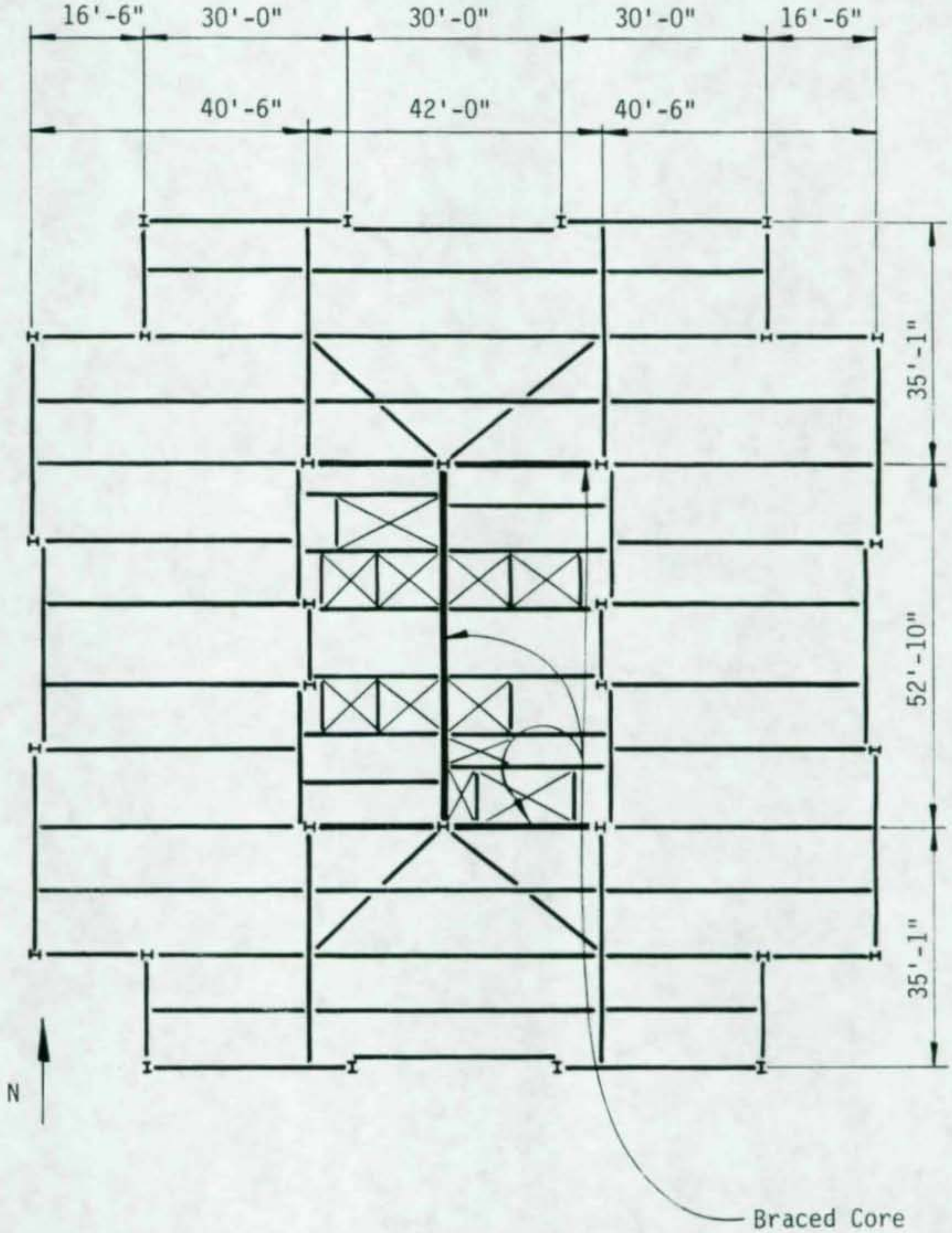


Figure 18 High Rise Office Building
Tower Floor Plan

Figure 19 details an isometric of one story of the braced core. The east-west forces are resisted by K-braces in the north and south walls of the core. The single north-south K-brace utilizes the K-braces at 90° as flanges with all six major core columns to help limit deflection. The other advantage of having the east-west K-braces separated is for resisting unbalanced torsional forces due to wind and/or stability bracing.

The three dimensional space frame was analyzed and designed to account for secondary moment effects because of the massiveness of the brace connections to the beams and columns.

The floor loading consists of:

Live Loads: 100 psf *

Dead Loads:

Concrete structural slabs:		60 psf
Ceiling/mechanical:		5 psf
	Total	165 psf (unreduced)
		105 psf (reduced)

Facade: Granite clad precast concrete panels 90 psf

* Reduced per code for design of beams, girders, columns and foundations (includes partitions per ANSI A58.1)

Design Comparison

The floor framing, braces and columns were designed by ASD and LRFD for the purposes of comparison. Table 3 itemizes the resulting size and cost differences.

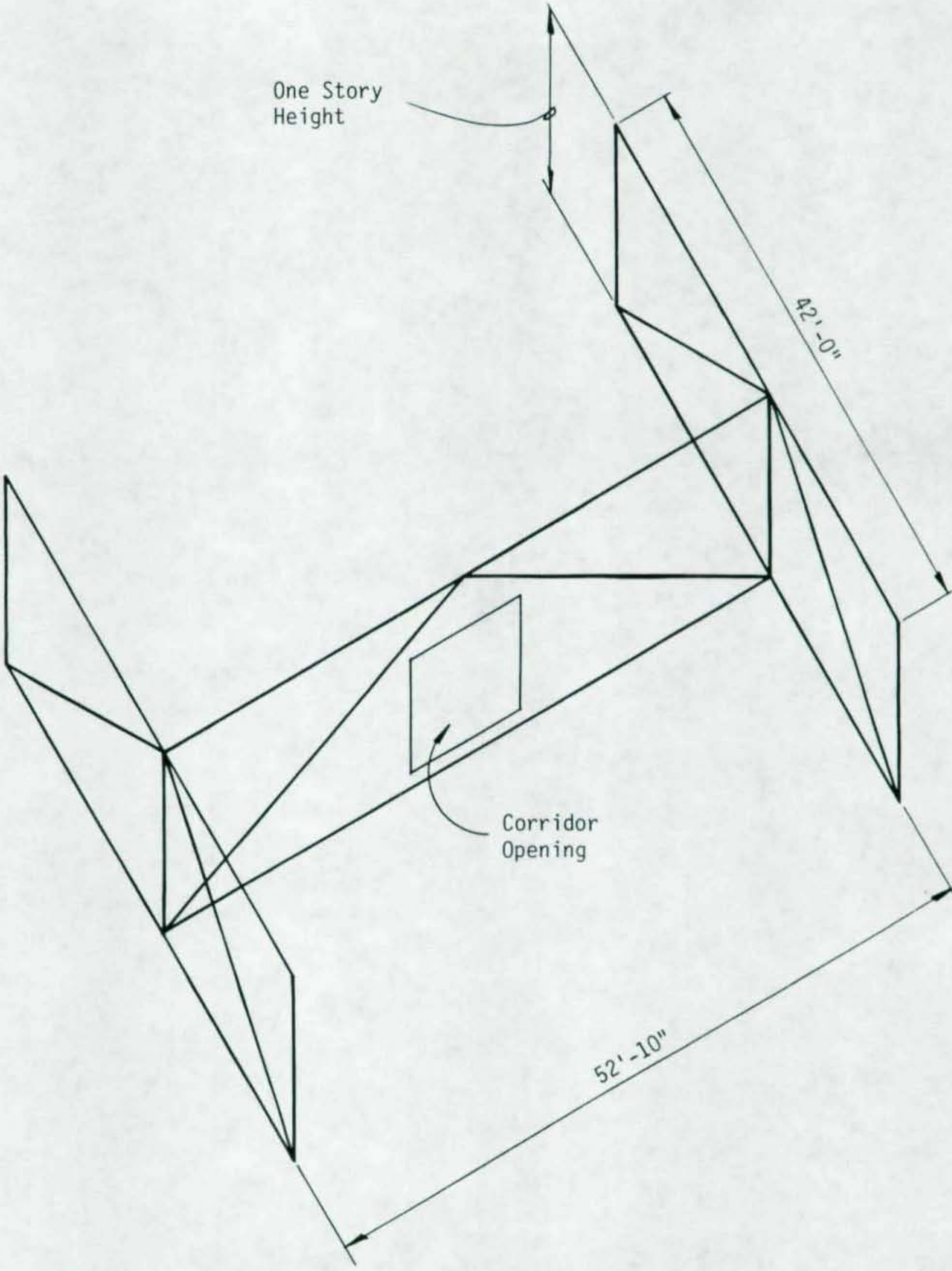


Figure 19 Isometric of K-Braced Core
(Typical Floor)

Table 3 Comparison of the High Rise Office Building Components Designed by ASD and LRFD

	ASD	LRFD
Beams (Cambered) Size (F_y) Studs Fabricated Cost Savings	 W21 x 44 (50)* 24 $684.00 -	 W21 x 44 (36)* 36 $647.00 5.5%
Spandrel Beams Size (F_y) Studs Fabricated Cost Savings	 W16 x 26 (50) 12 $274.25 -	 W16 x 26 (36) 12 $252.50 8%
Spandrel Girders Size (F_y) Studs Fabricated Cost Savings	 W21 x 44 (50) 39 $492.00 -	 W21 x 44 (36) 41 $458.00 7%
Spandrel Girders Size (F_y) Studs Fabricated Cost Savings	 W21 x 50 (50) 33 $542.25 -	 W21 x 44 (50) 25 $471.00 13%
Transfer Girders Size (F_y) Studs Fabricated Cost Savings	 W30 x 99 (50) 33 $1190.00 -	 W24 x 68 (50)* 126 $1043 12%
Spandrel Girder Size (F_y) Studs Fabricated Cost Savings	 W24 x 55 (50) 24 $578.00 -	 W21 x 44 (50) 40 $493.50 14.5%
Girders at Core Size (F_y) Studs Fabricated Cost Savings	 W18 x 35 (50) 50 $310.75 -	 W18 x 35 (50) 21 $267.25 14%
Girder at Core Size (F_y) Studs Fabricated Cost Savings	 W16 x 50 (50) 50 $411.75 -	 W16 x 36 (50) 66 $341.50 17%

Table 3 continued

	ASD	LRFD
K Brace Members (N-S) Size (F_y) To Size (F_y) Tonnage Weight Fabricated Cost Savings	 W14 x 145 (50) W14 x 90 (36) 62T $39,620 10.5%	 W14 x 176 (50) W14 x 90 (36) 68T $44,220 -
K Brace Members (E-W) Compared from 1st floor to 4th floor) Size (F_y) To Size (F_y) Tonnage Weight Fabricated Cost Savings	 W14 x 120 (50) W14 x 68 (50) 9.1 T $5705 12%	 W14 x 145 (50) W14 x 82 (50) 9.9 T $6491 -
Columns (Gravity Load) Tonnage Fabricated Cost Savings	 66.9T $43,920 -	 60.5T $39,670 9.5%
Columns (Gravity + Wind Load) Tonnage Fabricated Cost Savings	 70.9T $46580 3% **	 73.1T $48,030 -

* These beams require cambering to minimize "ponding" of concrete on deck. Cambering cost included in fabricated cost.

** 3% savings with ASD is an average. This percentage savings was 10% in the lower stories where column wind axial forces are 50% of the total load.

A study of the resulting designs discloses that:

1. Floor framing savings varied from 5.5% to 17%, with LRFD always less costly than ASD.

2. Minimum depth floor beams for deflection controlled many of the W21 and W24 beams. For example, the typical W21 beam could have been a W16 or W18 with LRFD with even greater cost savings, but the fabricator was not sure he would be able to camber them sufficiently. (Ref. 26).

3. K braces were larger with LRFD because the average load factor is in the range of 1.2 to 1.3. (See Fig. 4 which illustrates the effect of small amounts of gravity load on the load combination).

4. Gravity loaded columns were 9.5% lighter in tonnage with a LRFD procedure than ASD. The average LRFD load factor was about 1.3 which will always quantify a savings over ASD (see Fig. 6).

5. The core columns which have a substantial wind load axial force were heavier with LRFD procedures than ASD. The lowest story required a W14 x 605 (50) for LRFD and a W14 x 550 (50) for ASD. The wind force in these columns is substantially less in the upper stories with gravity load controlling the sizes.

Vibration was also checked for the open office area. This building did not have a vibration controlled minimum design for the LRFD because the open office area had beams and girders with spans of 35' to 40'. The susceptible range (in the writer's experience) of 20' to 32' is in areas where exterior wall systems or core wall systems will dampen potential vibrations.

Summary of Design Studies

The tabulated data illustrates all the major points disclosed in the previous chapters. These points can be summarized by the following:

1. Load Factors - Proportions of various types of loads will significantly impact final factor of safety and any comparison with ASD. All three building studies showed that normal design office gravity controlled LL/DL ratios will be less than the AISC LRFD calibration levels.

2. Column Design - The major column design change with LRFD (besides calibration factor) is the elimination of the special consideration for secondary members which has been in the AISC specification since its 1st Edition (Ref. 3).

3. Beam Design - Flexural design has undergone an increase in capacity due to the plastic section modulus and more liberal strength capacities at unbraced lengths beyond L_u.

4. Composite Beam Design - The ultimate strength approach to flexural design of composite beams increases LRFD capacities well beyond ASD capacities for virtually all load factors. A rule of thumb found in these design studies was that a composite ASD beam with F_y = 50 has about the same capacity as a composite LRFD beam with F_y = 36 and approximately the same number of shear studs.

5. Economy - LRFD will redistribute the tonnage in steel buildings as compared to ASD. Less structural steel will be in the gravity-controlled members and more in the wind-controlled members.

6. PR Connections - Combining Semi-Rigid and "Pinned" connections (Type 2 and 3, ASD) results in a better understanding that virtually all connections have restraint.

   It will be difficult to incorporate these provisions until accurate moment rotation characteristics can be determined at the design stage. And of course it will be important that the designer insures that the connection is built per his assumptions.

CHAPTER VIII

DISCUSSION AND CONCLUSIONS

The major advantages of AISC Load and Resistance Factor Design include:

1. Economy - In all normal gravity load control cases checked by the writer, LRFD components had more effective strength at service loads than ASD components.

2. Reliability Index - LRFD has the potential to give a more uniformly safe structure. The choice of load factors and ϕ factors <u>can</u> proportion members more closely to a given β level for either a strength or a serviceability limit boundary.

3. Continuity - The use of ANSI A58.1-82 load factors for steel design should encourage other material specification writing groups to incorporate the same format. This will simplify practical design efforts and coordination.

4. Design Philosophy - LRFD should challenge (and require) the designer to think and be more in tune with the loading and the structure. For example:

a. Classification of superimposed loading into dead load or live load may hinge upon uncertainties of loading and also whether to put certain loadings into load combinations with load factors less than 1.0.

b. The designer may view the LRFD calibration not appropriate for a certain type of structure or occupancy and may increase the load factors to compensate for this.

The writer endorses the LRFD methodology as a very rational design tool and will use it exclusively for design of all steel structures. He is, though, concerned about the selection of the appropriate load factors, ϕ factors, β index and limit state criteria. These concerns include:

1. The calibration of this specification has been to a resulting live load to dead load ratio which is higher than is normal for typical design office work. This effectively reduces the factor of safety from what is presently used (for gravity-controlled designs).

   The writer understands that the Metal Building Manufacturers Association lobbied successfully to have the calibration LL/DL ratio raised to these higher values based upon their experience (Ref. 15). It is the writer's opinion that light duty roofs which MBMA is using as an example of successfully executed and performing with a high LL/DL ratio, also have the greatest number of failures.

2. The writer understands the philosophy that low LL/DL ratio designs in ASD have a higher factor of safety than a higher LL/DL ratio design. This is because, theoretically, dead loads are better known in magnitude and location than live loads. Whether the previous statement is true or not is certainly open to discussion and is not the subject of this study. The net result of high LL/DL calibration of LRFD is an increased probability that a low LL/DL LRFD designed component will reach maximum strength at working loads.

3. The designer has lost a substantial portion of the benefit of the wind reduction factor with AISC and ANSI A58.1-82. This includes:

    a. The allowable stress load combination dictates that the 0.75 reduction factor only applies when wind is combined with both dead and live loads. AISC ASD 8th Edition still permits the 0.75 factor on wind control situations whether wind acts alone or in combination with other load.

b. ANSI does not recognize direction reduction and the probability density factor which has been proposed to address the probability of critical wind direction occurring simultaneously with critical pressure coefficients. (Ref. 24). This, if adopted (as was done by Canada for their limit state design code) would help mitigate the effect of item 3a.

c. The AISC LRFD and ASD 9th Edition which is to be published in 1989 has deleted the wind bracing member provision for a higher allowable stresses for slender members (Ref. 15).

The writer believes that the code and specification committee's decisions with respect to wind related forces are rational, but the profession must be aware that this will cause a net increase in the factor of safety for wind-controlled members.

4. There is still debate over the selection of LRFD load factors and ϕ factors in the literature. These include:

   a. Sulyok and Galambos propose that the ϕ factor for flexure be variable with a maximum of about 0.9 (present LRFD value) and a minimum of 0.81 (Ref. 25).

   b. Bennett proposal that the load factor for snow loads be increased from 1.6 (present LRFD value) to 2.0 to obtain a β factor of 3.0 (Ref. 6).

c. Bjorhovde has shown that LRFD column equations do not address the low β factors at intermediate slenderness nor can one set of equations cover all types of columns (Ref. 7).

5. Serviceability - The designer will need to be more cognizant of serviceability criteria such as deflection, vibration, and ponding. These particular limit states are being exceeded at a unacceptable frequency with ASD. The adoption of LRFD with its lighter member designs could increase serviceability failures.

The LRFD methodology study has raised many questions about the appropriateness of the ANSI load factors for normal design office projects. It has been shown that the level at which AISC calibrated the LRFD specification to the existing ASD specification gives 10% to 20% more capacity for LRFD, at what the writer considers normal ratios of LL/DL.

The writer proposes that the gravity-controlled load factor curve be changed to:

1.4 DL + 1.6 LL

Figure 20 is a comparison of ANSI (1.4 DL and 1.2 DL + 1.6 LL) with the proposed equation. It is shown that at a LL/DL ratio of 3, only a 3.3% increase in load factor occurs with the proposed equation over the existing ANSI equations.

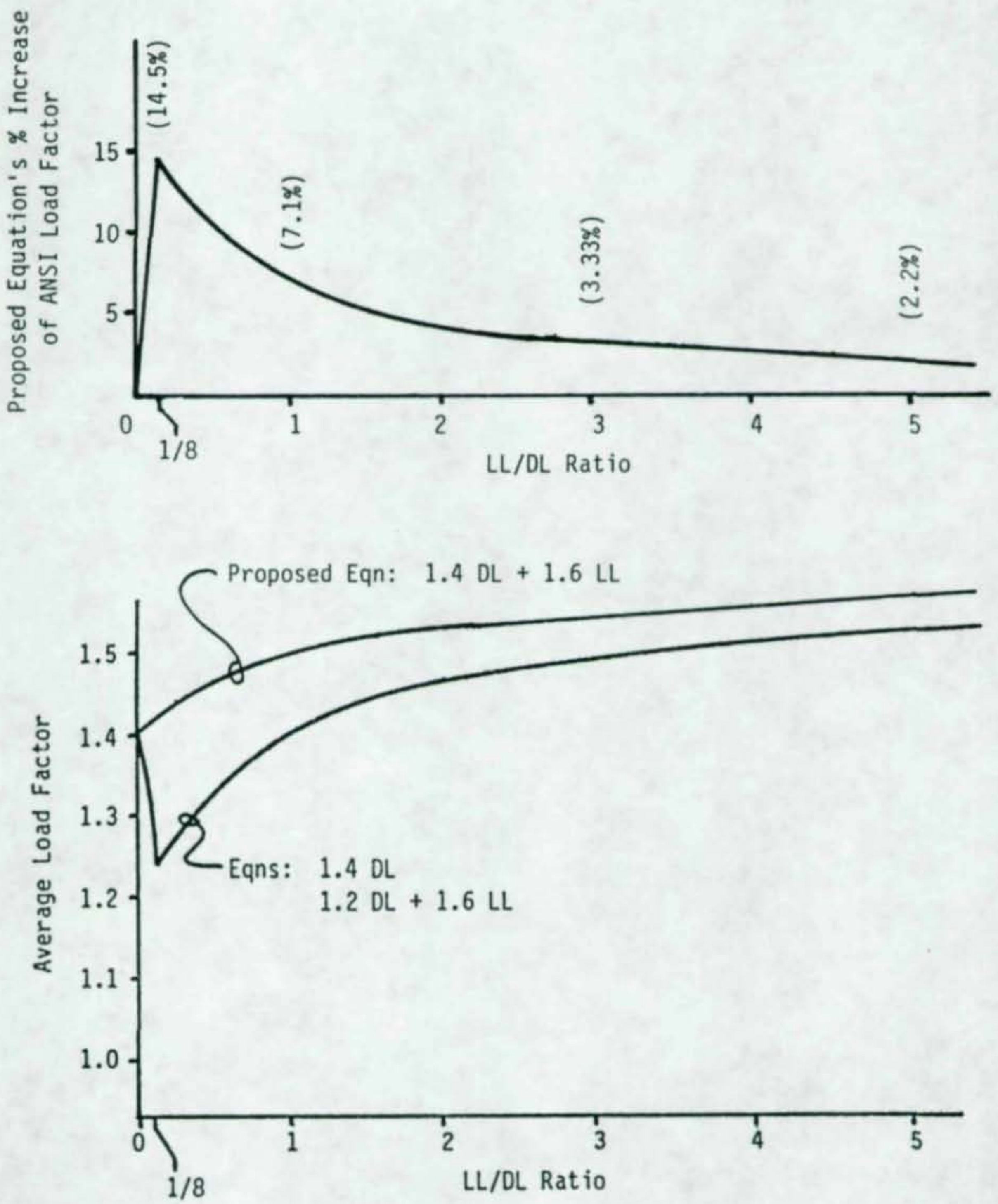


Figure 20 Load Factor Equations' Comparison

This proposed equation (1.4 DL + 1.6 LL) eliminates the illogical dip in the ANSI load factor response curve and will give a better ASD/LRFD calibration at commom LL/DL ratios.

The AISC Load and Resistance Factor Design methodology is rational and the state-of-the-art structural steel design. This study has shown that significant changes will develop in the final designed product as compared with AISC Allowable Stress Des·gn. Since this is AISC's first edition for LRFD, the writer believes that we will see further changes in this method before LRFD is adopted whole heartedly by the profession.

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