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Seismic Design Practice For Eccentrically Braced Frames

Based On The 1994 UBC

By

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&

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Contributors

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His work currently spans a wide range of structural design projects including eccentrically braced framed buildings. Prior to establishing his own firm, he was an Associate with Ove Arup & Partners in London and Los Angeles where the original work for this guide was written.

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These structures have varied from high-rise office building almost 700 feet in height, to 300 foot clear span convention centers and aircraft hangars, to Titan missile launching facilities. While most of these structures are located in California, a significant number are located in such distant locations as Saudi Arabia and Diego Garcia where unique construction requirements were necessary.

At the present time, Mr. Becker is a principal of the firm Becker and Pritchett Structural Engineers, Inc. which is located at Lake Forest, California. Before establishing his own firm, he was Chief Structural Engineer for VTN Consolidated Inc. He also served as Regional Engineer in Los Angeles for the American Institute of Construction, Inc. Prior to this, he was engaged with the Los Angeles engineering firm of Brandow & Johnston Associates.

He has authored the following seismic design publications for steel construction:

- "Practical Steel Design for Building 2-20 Stories," 1976.
- "Seismic Design Practice for Steel Buildings," 1988.
- "Seismic Design of Special Concentrically Braced Frames," 1995.

PREFACE

This booklet is an update and revision of the Steel Tips publication on eccentrically braced frames dated May 1993 (ref. 16).

The significant revisions to the May 1993 booklet are as follows:

- Design criteria is based on the 1994 Edition of the Uniform Building Code.

- The steel for the link beam element has a yield strength of 50 ksi. Based on current mill practices, this yield strength should be utilized for the capacity of the link beam for A36, A572 grade 50 and Dual Grade Steels.

- The use of a link adjacent to a column is not "encouraged." This is due to the moment connection required at the beam to column intersection and the possible difficulty in achieving a moment connection which can accommodate large rotations of the link subject to high shear and moment without significant loss of capacity. See Ref. 11 p. 333 for additional information.

- The beam outside the link has a strength at least 1.5 times the force corresponding to the link beam strength.

It should be noted that ASTM and the Structural Steel Shapes Producers Council are in the process of writing a proposed "Standard Specification for Steel for Structural Shapes used in Building Framing." At the present time, this single standard would require that the following be met: yield strength = 50 ksi MIN; tensile strength = 65 ksi MIN; yield to tensile ratio = 0.85 MAX. However, these requirements are still under discussion and negotiation, but hopefully this single standard will be published by ASTM in the next year or two.

CONTENTS

SYMBOLS AND NOTATIONS

A_f	Area of a flange $A_f = b_f t_f$, in.^{2}
A_w	Cross sectional area of column or beam web $A_w = t_w d$, in.^{2}
a	Beam length between a column and a link, in.
a	Weld size, in.
a'	Maximum allowable unbraced length for the flanges of a link, in.
b	Stiffener plate width, in.
b_f	Flange width, in.
C	Code lateral force coefficient, used with other factors in base shear formula
C_1	Lateral force coefficient equal to V/W
C_m	Bending interaction coefficient
C_t	Period mode shape constant
d	Beam depth, in.
e	Eccentricity between the center of mass and the center of rigidity, feet
e	Link length, in.
e'	Recommended length for shear links, $e = 1.3\, M_s / V_s$ in.
F_a	Allowable compressive stress, ksi
F_e'	Euler stress for a prismatic member divided by a factor of safety, ksi
F_i	Code lateral force at level i, kips
F_t	Code lateral force at top of structure, kips
F_x	Code lateral force at level x, kips
F_y	Specified minimum yield stress of steel, ksi
F_w	Allowable shear stress in a weld, ksi
f_a	Actual compressive stress, ksi
f_i	Applied lateral force at i, kips
g	Acceleration of gravity, 386 in./sec.^{2}
h	Building height above rigid base
h	Frame height (c-c beams)
h_c	Clear height of column
h_i	Building height to level i
h_n	Building height to level n
I	Importance factor related to occupancy used in lateral force formula
I_x	Strong axis moment of inertia of a steel section, in.^{4}
k	Kip (1000 lbs. force)
klf	Kips per linear foot
ksi	Kips per square inch
L	Beam length (c-c columns), in.
L_c	Beam clear length between columns
LF	Plastic design load factor
l	Weld length, in.
M_{be}	Moment in a beam from an elastic analysis, in. kips
M_{bu}	Factored design moment in the beam outside the link, in. kips
M_{ce}	Moment in a column from an elastic analysis, in. kips
M_{cu}	Factored design moment in the column, in. kips
M_{lU}	Factored design moment in the link, in. kips
M_m	Maximum moment that can be resisted in the absence of axial load, in. kips
M_P	Plastic moment, in. kips

M_{rs} Link flexural capacity reduced for axial forces $M_{rs} = Z_x(F_y - f_a)$ or $M_{rs} = Z_f(F_y - f_a)$. in. kips
M_s Member flexural strength $M_s = Z_x F_y$, in. kips
M_{VERT} Moment in a link from gravity load, in. kips
n The uppermost level in the main portion of the structure
P Vertical load on column, kips
P_{br} Factored design compression in the brace, kips
P_{bu} Factored design compression outside the link, kips
P_{cr} Strength of an axially loaded compression member, kips
P_{cu} Factored design compression in the column, kips
P_{ot} Axial column load due to seismic overturning, kips
P_E Axial load on a member due to earthquake
P_e Euler buckling load, kips
P_l Unfactored link axial load, kips
P_{lu} Factored link axial load, kips
P_{sc} Axial compression strength of a member $P_{sc} = 1.7F_aA$, kips
P_y Plastic axial load $P_y = F_yA$, kips
R_w Numerical coefficient based on structural lateral load-resisting system
r_x Radius of gyration with respect to the x-x axis, in.
r_y Radius of gyration with respect to the y-y axis, in.
S Site structure coefficient
S_x Strong axis section modulus, in.^{3}
T Period of vibration for single degree of freedom systems. Fundamental (first mode) period for multiple degree of freedom systems, seconds
t Stiffener plate thickness, in.
t_f Flange thickness, in.
t_w Web thickness, in.
V Lateral force or shear at the base of structure, kips
V_b Beam shear reaction corresponding to V_s, kips
V_{br} Shear to be resisted by the brace, kips
V_g Shear from gravity loading, kips
V_l Untactored design shear force in the link, kips
V_{rs} Shear capacity required to accommodate M_{rs}, kips
V_s Link shear strength $V_s = 0.55F_y dt_w$, kips
V_{VERT} Shear force in a link from gravity load, kips
V_x Lateral force at level x, kips
W The total seismic dead load defined by Code, kips, or uniform total load applied to a beam
W_i That portion of W which is assigned to level i, kips
W_d Uniform dead load applied to a beam, klf
W_l Uniform live load applied to a beam, klf
Z Seismic zone factor used in the lateral force formula
Z_f Plastic modulus of the flanges $Z_f = (d-t_f)b_f t_f$, in.^{3}
Z_x Strong axis plastic modulus, in.^{3}
Δ Lateral displacement (at top of structure unless noted otherwise), in.
δ_i Horizontal displacement at level i relative to the base due to applied lateral forces, in.
δ_x Horizontal displacement at level x relative to the level below due to applied lateral forces, (story drift), in.
ϕ Link capacity excess factor
θ Rotation of the link relative to the brace, radians.

SECTION 1

INTRODUCTION TO ECCENTRICALLY BRACED FRAMES (EBFs)

1.1 Introduction

EBFs address the desire for a laterally stiff framing system with significant energy dissipation capability to accommodate large seismic forces (ref. 7). A typical EBF consists of a beam, one or two braces, and columns. Its configuration is similar to traditional braced frames with the exception that at least one end of each brace must be eccentrically connected to the frame. The eccentric connection introduces bending and shear forces in the beam adjacent to the brace. The short segment of the frame where these forces are concentrated is the link.

EBF lateral stiffness is primarily a function of the ratio of the link length to the beam length (ref. 8, p. 44). As the link becomes shorter, the frame becomes stiffer, approaching the stiffness of a concentric braced frame. As the link becomes longer, the frame becomes more flexible approaching the stiffness of a moment frame.

The design of an EBF is based on creating a frame which will remain essentially elastic outside a well defined link. During extreme loading it is anticipated that the link will deform inelastically with significant ductility and energy dissipation. The code provisions are intended to ensure that beams, braces, columns and their connections remain elastic and that links remain stable. In a major earthquake, permanent deformation and structural damage to the link should be expected.

There are three major variables in the design of an EBF: the bracing configuration, the link length, and the link section properties. Once these have been selected and validated the remaining aspects of the frame design can follow with minimal impact on the configuration, link length or link size.

Identifying a systematic procedure to evaluate the impact of the major variables is essential to EBF design. It care is not taken to understand their impact, the designer may iterate through a myriad of possible combinations. The strategy proposed in this guide is to:

1) Establish the design criteria.
2) Identify a bracing configuration.
3) Select a link length.
4) Choose an appropriate link section.
5) Design braces, columns and other components of the frame.

EBF design, like most design problems, is an iterative process. Most designers will make a preliminary configuration, link length and link size selection based on approximations of the design shears. Reasonable estimates for braces and columns can easily follow. Once preliminary configurations and sizes are identified, it is anticipated that the designer will have access to an elastic analysis computer program to use in refining the analysis of the building period, the base shear, the shear distribution within the building, the elastic deflection of the structure and the distribution of forces to the frame members.

1.2 Bracing Configuration

The selection of a bracing configuration is dependent on many factors. These include the height to width proportions of the bay and the size and location of required open areas in the framing elevation. These constraints may supersede structural optimization as design criteria.

UBC 2211.10.2 requires at least one end of every brace to frame into a link. There are many frame configurations which meet this criterion.

1.3 Frame Proportions

In EBF design, the frame proportions are typically chosen to promote the introduction of large shear forces in the link. Shear yielding is extremely ductile with a very high inelastic capacity. This, combined with the benefits of stiff frames, make short lengths generally desirable.

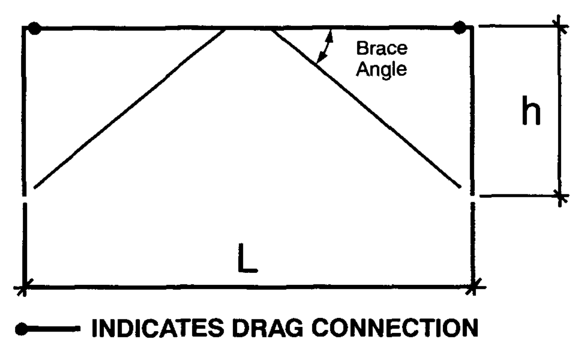


Figure 1. Frame Proportions

Keeping the angle of the brace between 35° and 60°, as shown in Figure 1, is generally desirable. Angles outside this range lead to awkward details at the brace-to-beam and brace-to-column connections. In addition to peculiar gusset plate configurations, it is difficult to align actual members with their analytic work points. Small angles can also result in an undesirably large axial force component in the link beams (ref 9, p. 504)

For some frames, the connection of the brace at the opposite end from the link is easier if a small eccentricity is introduced. This eccentricity is acceptable if the connection is designed to remain elastic at the factored brace load.

Optimizing link design requires some flexibility in selecting the link length and configuration. Accommodating architectural features is generally easier in an EBF than in a concentrically braced frame. Close coordination between the architect and engineer is necessary to optimize the structural performance with the architectural requirements.

1.4 Link Length

The inelastic behavior of a link is significantly influenced by its length. The shorter the link length, the greater the influence of shear forces on the inelastic performance. Shear yielding tends to happen uniformly along the link. Shear yielding is very ductile with an inelastic capacity considerably in excess of that predicted by the web shear area, provided the web is adequately braced against buckling (ref. 9, p. 499; ref. 10, p. 73).

Links usually behave as short beams subjected to equal shear loads applied in opposite directions at the link ends. With this type of loading, the moment at each end is equal and in the same direction. The deformation of the link is an S shape with a point of counterflexure at midspan. The moment is equal to 1/2 the shear times the length of the link.

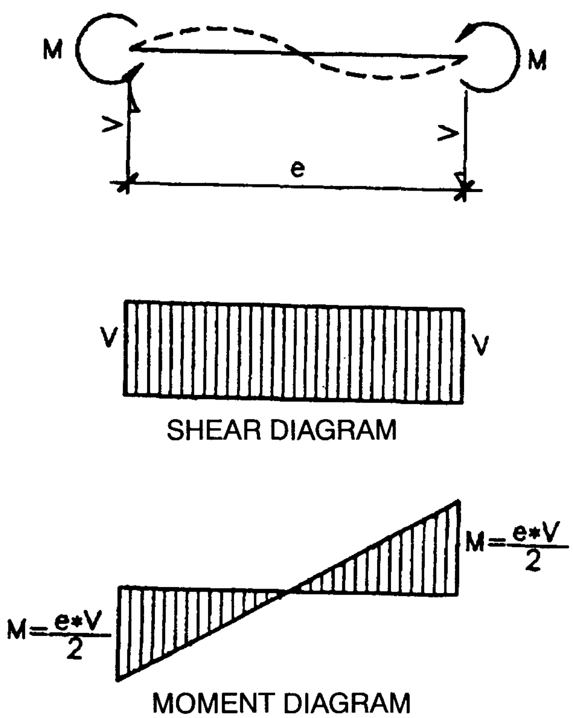


Figure 2.
Typical Link Loading

Link lengths generally behave as follows:

$e < 1.3 \frac{M_s}{V_s}$ assures shear behavior, recommended upper limit for shear links (ref. 8, p. 46)

$e < 1.6 \frac{M_s}{V_s}$ link post - elastic deformation is controlled by shear yielding. UBC2211.10.4 rotation transition. (ref. 11, p. 331, C709.4)

$e = 2.0 \frac{M_s}{V_s}$ link behavior is theoretically balanced between shear and flexural yielding

$e < 2.0 \frac{M_s}{V_s}$ link behavior considered to be controlled by shear for UBC 2211.10.3 (ref 11, p. 330, C709.3)

$e > 3.0 \frac{M_s}{V_s}$ link post - elastic deformation is controlled by flexural yielding. UBC2211.10.4 rotation transition. (ref. 11, p. 331, C709.4)

Note: Most of the research to date has been on link lengths less than 1.6 M_s/V_s. These links generally behave well, exhibiting high ductility with good stability in the hysteretic response.

The shorter a link length is, the greater the rotation of the link will be. UBC 2211.10.4 sets limits on these rotations. When these limits are exceeded, the lateral deflection must be reduced or the link length increased. For most designs, link lengths of approximately 1.3 M_s/V_s work well (ref. 8, p. 46). This allows the designer some flexibility to change member sizes and link lengths during the design process and still remain below the 1.6 M_s/V_s code cutoff for shear links. Keeping link lengths near the upper limit of shear governed behavior generally results in acceptable link rotation.

Selection of link length is often restricted by architectural or other configuration restraints. In the absence of restraints, preliminary link length estimates of 0.15L for chevron configurations are reasonable.

The excellent ductility of shear yielding prompts most designers to use shear links. When the minimum link length is restricted, cover plates may be added to the flanges to increase the flexural capacity and transform a moment link into a shear link, or the link beam can be fabricated as a built up section from plates. Plastic deformation of the link will cause a discontinuity in the deflection curvature of the beam. This is likely to concentrate structural and non-structural damage around the link.

1.5 Link Beam Selection

Link beams are typically selected to satisfy the minimum web area required to resist the shear from an eccentric brace. It is generally desirable to optimize the link selected to meet but not exceed the required dt_w. Excess web area in the link will require oversizing other components of the frame, as they are designed to exceed the strength of the link.

Shear deformation in the link usually makes a modest contribution to the elastic deformation of a frame. Elastic deflection is dominated by the bending of the beams and columns and by axial deformation of the columns and braces. Inelastic deformation of the frame is dominated by rotation of the link caused by its shear deformation. Consequently, the link beams which appear the stiffest in an elastic analysis do not necessarily have the greatest ultimate shear capacity. The elastic contribution of shear to lateral deflection is tabulated for an example frame in Section 3.4, "Elastic Analysis".

Generally the design of a link beam is optimized by selecting a section with the minimum required shear capacity and the maximum available bending capacity. The most efficient link sections are usually the deepest sections with the minimum required shear area which comply with the compact web requirements of UBC Chapter 22, Division IX, Table B5.1, and meet the flange width-thickness ratio, $b/2t_f$, not exceeding $52/\sqrt{F_y}$. When the depth or flange size is restricted, the designer may wish to select a section which complies with the shear requirements and add cover plates to increase the flexural capacity. Cover plates may also be used to increase the flexural capacity and transform a bending link into a shear link when non structural restrictions prevent reducing the link length. The designer may customize the section properties by selecting both the web and flange sizes and detailing the link as a built up section.

1.6 Link Beam Capacity

Since the link portion of the beam element is the "fuse" that determines the strength of other elements, such as the braces and columns, its capacity should be conservatively determined based on the actual yield strength of the material.

Based on current mill practices, the yield strength of A36 material is approaching 50 ksi, and it will exceed 50 ksi if it is produced as a Dual Grade Steel meeting both A36 and A572 Grade 50 requirements.

Thus, it is now recommended that the capacity of the link beam should be based on a yield strength of 50 ksi for A36, A572 Grade 50 and Dual Grade Steels. Although the actual yield point may somewhat exceed 50 ksi, this has been accounted for in the over-strength factors of 1.25 and 1.50 required for the columns and braces, respectively, of the EBF frame.

SECTION 2

DESIGN CRITERIA FOR A 7-STORY OFFICE BUILDING

The example building has been selected to resemble the example previously used in "Seismic Design Practice for Steel Buildings" (ref. 5). The interior bay spacing has been modified to provide height to span proportions better suited for EBFs. All other design parameters have been retained.

The building will be designed in accordance with the 1994 Edition of the Uniform Building Code (ref. 2). Seismic design is based on Chapter 16, Division III essentially the same as the 1996 "Recommended Lateral Force Requirements," of the Structural Engineers Association of California (ref. 11 Chapter 1).

Design of steel members and connections is based on *Section 2211* & Chapter 22 of the 1994 UBC (ref. 2) & Ref. 11.

The building is located in Seismic Zone No. 4. The geotechnical engineer has determined that the soil profile consists of a dense soil where the depth exceeds 200 feet.

The frame is to be structural steel. As shown in Figure 3, it has Chevron eccentric braced frames in the N-S direction on column lines 1 and 6. Chevron EBFs are provided in the E-W direction, along column lines A and D. Floors and roof are 3" metal deck with 3-1/4" lightweight (110 pcf) concrete fill. Typical story height is 11'-6", based on 8'-0" clear ceiling height.

In this example the EBFs are only one bay wide. This concentrates the overturning moment in adjacent columns resulting in extreme axial compression and tension for the column and foundation design. While this is convenient to illustrate the impact of shear link capacity on the column design, it may not provide the best building solution. Often overall economy is achieved by spreading the overturning to the outside columns. This reduces the overturning axial compression and tension in the columns. Unless there is a basement or other significant load distribution mechanism below grade, the foundations can get very large to support a narrow frame with its correspondingly high soil reactions.
Material specifications are:

Steel beams: ASTM A572 Grade 50, $F_y = 50$ ksi
Steel braces: ASTM A500 Grade B, $F_y = 46$ ksi
Steel columns: ASTM A572 Grade 50, $F_y = 50$ ksi
High-strength bolts: ASTM A325
Welding electrodes: AWS E70XX

2.1 Loads

<u>Roof Loading</u>:	
Roofing and insulation	7.0 psf
Metal deck	3.0
Concrete fill	44.0
Ceiling and mechanical	5.0
Steel framing and fireproofing	8.0
Dead Load	67.0 psf
Live load (reducible), UBC 1605.1	20.0
Total Load	87.0 psf
<u>Floor Loading</u>:	
Metal deck	3.0 psf
Concrete fill	44.0
Ceiling and mechanical	5.0
Partitions, UBC 1604.4	20.0
Note: The partition load could be reduced to 10 psf for lateral analysis, UBC 1628.1	
Steel framing, incl. beams, girders, columns, and spray-on fireproofing	13.0
Dead Load	85.0 psf
Live load (reducible) UBC 1604.1	50.0
Total Load	135.0
<u>Curtain wall:</u>	
Average weight	15.0 psf

2.2 Base Shear Coefficient

$$V = \left(\frac{ZIC}{R_w}\right) W \qquad \text{UBC (28-1)}$$

$$C = \frac{1.25S}{T^{2/3}} \qquad \text{UBC (28-2)}$$

$Z = 0.4$ UBC Table 16-I
$I = 1.0$ UBC Table 16-K
$R_w = 10.0$ UBC Table 16-N
$S = 1.2$ UBC Table 16-J

$$V = \frac{0.4\ (1.0)C}{10} W = 0.040CW$$

C, and therefore *V*, is a function of *T*, the fundamental period of vibration. The building period must be estimated before *V* can be calculated.

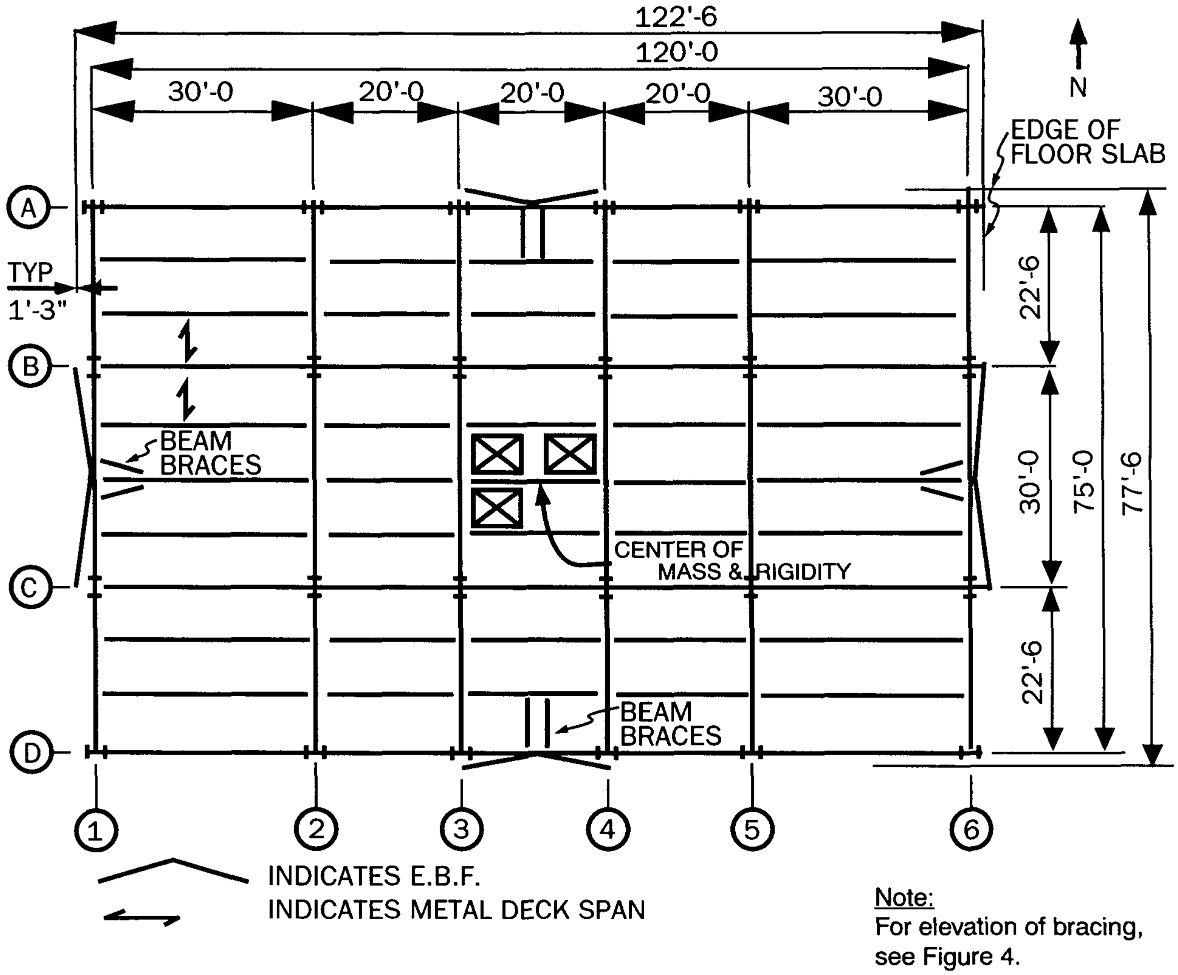


Figure 3.
Framing Plan

The UBC recognizes two methods for determining T. Method A is based on the building height and the type of lateral system. Method B requires an estimate of the lateral load distribution and the corresponding deflections. Method B provides greater insight into the behavior of the building and should be used at some point during the design process. UBC 1628.2.2 limits the fundamental period to 130% of that obtained from Method A. With this limitation, the base shear lower limit would be 84% of that obtained from Method A.

For most frames, the building period calculated by Method B is significantly longer than from Method A. Consequently, the 84% of the Method A base shear lower limit often governs the strength design of frame structures. This lower limit does not apply to deflection governed structures per UBC 1628.8.3. Most low and medium height buildings with shear link EBFs are governed by strength. For tall structures, or EBFs with moment links, drift control typically governs the design. **Both strength and deflection criteria must be checked in all designs.**

In this seven story shear link frame, strength will probably govern the design. The base shear calculated by Method B will probably be less than 84% of the base shear calculated by Method A. Consequently, 84% of the Method A base shear will be distributed in each direction. Members will be sized for this shear. These members will be used in an elastic computer analysis to determine: the deflection of the frames, the relative rigidity of the E-W frames, and the building period. This information can then be used to refine the design shear and corresponding frame sections if necessary.

2.3 Building Period and Coefficient C

Using Method A,

$T = C_t\ (h_n)^{3/4}$ UBC (28-3)

$C_t = 0.030$ for EBFs UBC 1628.2.2

$h_n = 83.0$ ft.

$T = 0.030(83.0)^{3/4} = 0.825$ seconds

Note: $T > 0.7,\ F_t \neq 0$ UBC 1628.4

$C = \dfrac{1.25(1.2)}{(0.825)^{2/3}} = 1.71$

Note: $C < 2.75$ ∴ o.k. UBC 1628.2.1

$\dfrac{C}{R_w} = \dfrac{1.71}{10} = 0.171$

Note: $\dfrac{C}{R_w} > 0.075$ ∴ o.k. UBC 1628.2.1

Using Method B,

$T_{METHOD\ B} = 1.3\ T_{METHOD\ A}$ *(Maximum for Stress) per UBC 1628.2.2*

$C_{METHOD\ B} = \dfrac{1.25\ S}{(1.3T)^{2/3}}$

$= 0.84\ \dfrac{1.25\ S}{T^{2/3}}$

$= 0.84C_{METHOD\ A}$

Therefore the minimum base shear obtained by Method B is 84% the base shear calculated by Method A.
For frame stress analysis use:
Value of C determined from T of Method B

$T = 1.3 \times 0.825 = 1.073$ seconds

$C_{METHOD\ B} = \dfrac{(1.25)\ (1.2)}{(1.073)^{2/3}} = 1.43$

Note: When the sizes of the braced frame members have been determined, the period should be found using Method B, UBC (28-5). For the assumed strength criteria to be valid ($C_{METHOD\ B}$ *= 1.43):* $T_{Method\ B} \geq 1.073$ seconds assures that the design base shear for stress will be governed by using 84% of the base shear resulting from calculating the building period using Method A.

2.4 Design Base Shear and Vertical Distribution

$V = 0.04CW$ *(per Section 2.2)*

$C_{METHOD\ B} = 1.43$ for stress calculations *(per Section 2.3)*

$V_{STRESS} = 0.040\ (1.43)\ W = 0.0572W$

$W_{fl} = (122.5\text{x}77.5)(.085)+(400\text{x}11.5)(.015)$
$= 807+69 = 876$ kips

$W_{rf} = (122.5\text{x}77.5)(.067)+(400\text{x}(11\ 5/2+3.0))(.015)$
$= 636+52 = 688$ kips

$W = 6(876)+688 = 5{,}940$ kips (total dead load)

$V_{STRESS} = 0.0572W = 340$ kips

The total lateral force is distributed over the height of the building in accordance with UBC Formulas (28-6),(28-7) and (28-8).

$V = F_t + \sum_{i=1}^{n} F_i$ UBC (28-6)

$F_t = 0.07TV = 2.6$ *kips* UBC (28-7)

$F_x = \dfrac{(V - F_t)\ w_x h_x}{\sum_{i=1}^{n} w_i h_i}$ UBC (28-8)

The distribution of lateral forces over the height of the building is shown in Table 1.

TABLE 1
Distribution of Lateral Forces

Level	h_x ft.	w_x kips	$w_x h_x$ x10^{-2}	$\frac{w_x h_x}{\Sigma w_i h_i}$	STRESS F_x (1) kips	STRESS V_x (1) kips
R	83.0	688	571	0.203	64+26[(2)] =90	—
7	71.5	876	626	0.222	70	90
6	60.0	876	526	0.187	59	160
5	48.5	876	425	0.151	47	219
4	37.0	876	324	0.115	36	266
3	25.5	876	223	0.079	25	302
2	14.0	876	123	0.043	13	327
1	—	—	—	—	—	340
Σ	—	—	2,818	1.000	340	—

(1) Forces or shears for use in stress calculations (min V= 84% from Method A).

(2) At roof, $F_x = (F_t + F_n)$

It is assumed that wind loading is not critical for lateral forces in this design example. If wind did control the design of the frame, it would be necessary to recalculate both the period and the earthquake forces based on the stiffness requirements of the frame to resist wind. Allowable wind drift is usually taken = 0.0025 times the story height.

2.5 Horizontal Distribution of Seismic Forces

Although the centers of mass and rigidity coincide, UBC 1628.5 requires designing for a minimum torsional eccentricity, e, equal to 5% of the building dimension perpendicular to the direction of force regardless of the relative location of the centers of mass and rigidity. To account for this eccentricity, many designers add 5 to 10% to the design shear in each frame and proceed with the analysis. For this example, numerical application of the code provisions will be followed.

$e_{ew} = (0.05)(75) = 3.75$ ft.
$e_{ns} = (0.05)(120) = 6.00$ ft.

Shear distributions in the E-W direction:

All four EBFs will resist this torsion.

Assume that all the frames have the same rigidity since all are EBFs. This assumption can be refined in a subsequent analysis, after members have been sized and an elastic deflection analysis has been completed.

$R_1 = R_6 = R_A = R_D = 1.0$

$$V_{A,x} = V_{D,x} = R_A \left[\frac{V_x}{\Sigma R_{e\text{-}w}} \pm \frac{(V_x e)(d)}{\Sigma R_y (d)^2} \right]$$

where

e = Torsional eccentricity

d = Distance from frame to center of rigidity

$R_{e\text{-}w}$ = Rigidity of those frames extending in the east west direction

R_y = Rigidity of a frame, referenced to column line *y* which is a perpendicular distance *d* from the center of rigidity

V_x = Total earthquake shear on building at story x

$V_{y,x}$ = Earthquake shear on an EBF referenced to that frame on column line y at story x

$\Sigma R_{e\text{-}w} = 2(1.00) = 2\ 0$

$\Sigma R_y d^2 = 2(1.00)(37.5)^2 + 2(1.\ 00)(60.00)^2 = 1\ 0{,}01\ 2$

$$V_{A,x} = 1.00 \left[\frac{V_x}{2.00} \pm \frac{(V_x \times 3.75)}{10{,}012} \right] = 0.500 V_x \pm 0.014 V_x$$

$$= 0.514 V_x$$

Shear distribution for north-south direction:

$$V_{1,x} = V_{6,x} = R_1 \left[\frac{V_x}{\Sigma R_{n\text{-}s}} \pm \frac{(V_x e)(d)}{\Sigma R_y (d)^2} \right] = V_{DX}$$

$\Sigma R_{n\text{-}s} = 2(1.00) = 2.0$

$\Sigma R_y d^2 = 2(1.00)(37.5)^2 + 2(1.00)(60.00)^2 = 10{,}012$

$$V_{1,x} = 1.00 \left[\frac{V_x}{2.00} \pm \frac{(V_x \times 6.00)(60.0)}{10{,}012} \right] = 0.500 V_x \pm 0.036 V_x$$

$$= 0.536 V_x$$

TABLE 2
Frame Forces

	East-West		North-South	
	EBF A & D (0.514 F_x)		EBF 1 & 6 (0.536 F_x)	
	STRESS		STRESS	
LEVEL	F_x kips	V_x kips	F_x kips	V_x kips
R	46	—	48	—
7	36	46	38	48
6	30	82	32	86
5	24	112	25	118
4	19	136	19	143
3	13	155	13	162
2	7	168	7	175
1	—	175	—	182
Σ	175	—	182	—

UBC 1631.2.9 specifies the diaphragm design loads. These are shown in Table 3.

TABLE 3
Diaphragm Design Loads

Lvl	w_i kips	Σw_i kips	f_i kips	Σf_i kips	w_{px}(1) kips	0.35ZI w_{px}(2) kips	F_{px}(3) kips	0.75ZI w_{px}(4) kips
R	688	688	90	90	688	96	90.0	206
7	876	1,564	70	160	876	123	89.6	263
6	876	2,440	59	219	876	123	78.6	263
5	876	3,316	47	266	876	123	70.3	263
4	876	4,192	36	302	876	123	63.1	263
3	876	5,068	25	327	876	123	56.5	263
2	876	5,944	13	340	876	123	50.1	263
Σ	5,944		340					

(1) w_{px}, the weight of the diaphragm and tributary elements, is taken as the roof or floor weight, w_i.

(2) Minimum allowed diaphragm design load. UBC 1631.2.9

(3) Diaphragm design load.

$$F_{px} = \left(\frac{\sum_{i=x}^{n} f_i}{\sum_{i=x}^{n} w_i} \right) w_{px} \qquad \text{UBC 1631.2.9 (31-1)}$$

(4) Maximum required diaphragm design load. UBC 1631.2.9

SECTION 3

CHEVRON CONFIGURATION / BEAM SHEAR LINK, EAST-WEST FRAME

3.1 Introduction

As indicated in Figure 4, the frame geometry and the lateral loads from Table 2 are sufficient to begin sizing the EBF members. It is not necessary to include the effect of gravity loads on beams and columns or to perform an elastic analysis before a reasonable estimate of the member sizes can be made. The designer may proceed directly to Section 3.6, "Link Size", and begin by sizing the top link in the frame and proceed down to the foundation.

To illustrate a design procedure which accounts for the influence of gravity load on the lateral system, the example will proceed by analyzing the suitability of these members at the first story, including second floor link beam, as indicated in Figure 4.

The frame member sizes shown in Figure 4 are the result of several design iterations using computer analysis.

3.2 Beam Gravity Loads

The beam does not need to be designed to support gravity loads presuming that the bracing does not exist, as required for chevron bracing in a concentric braced frame.

In EBFs which do not have transverse purlins framing into the beams, the influence of gravity load on the beam selection is usually not significant. Occasionally, the designer may wish to combine stress from these loads with the shear and bending stress resulting from the application of lateral load to the frame.

In Figure 5 the second level floor beam between grids 3 and 4 on grid line A or D is modeled. The section properties and link length shown in Figure 4 are used. To simplify the analysis the beam is assumed to have pinned ends.

For the second floor beam:

$$W_d = \left(\frac{22.5}{6}+1.25\right)(0.085)^{(1)} = 0.425 \text{ tributary floor}$$

$$+\left(\frac{11.5+14.0}{2}\right)(0.015) = \underline{0.192} \text{ tributary cladding}$$

$$0.616 \text{ klf}$$

$$W_l = \left(\frac{22.5}{6}+1.25\right)(0.050) = \underline{0.250}$$

$$W = \quad 0.866 \text{ klf} \quad \text{(total load)}$$

(1) 0.085 psf includes the estimated weight of girders and columns and is slightly conservative.

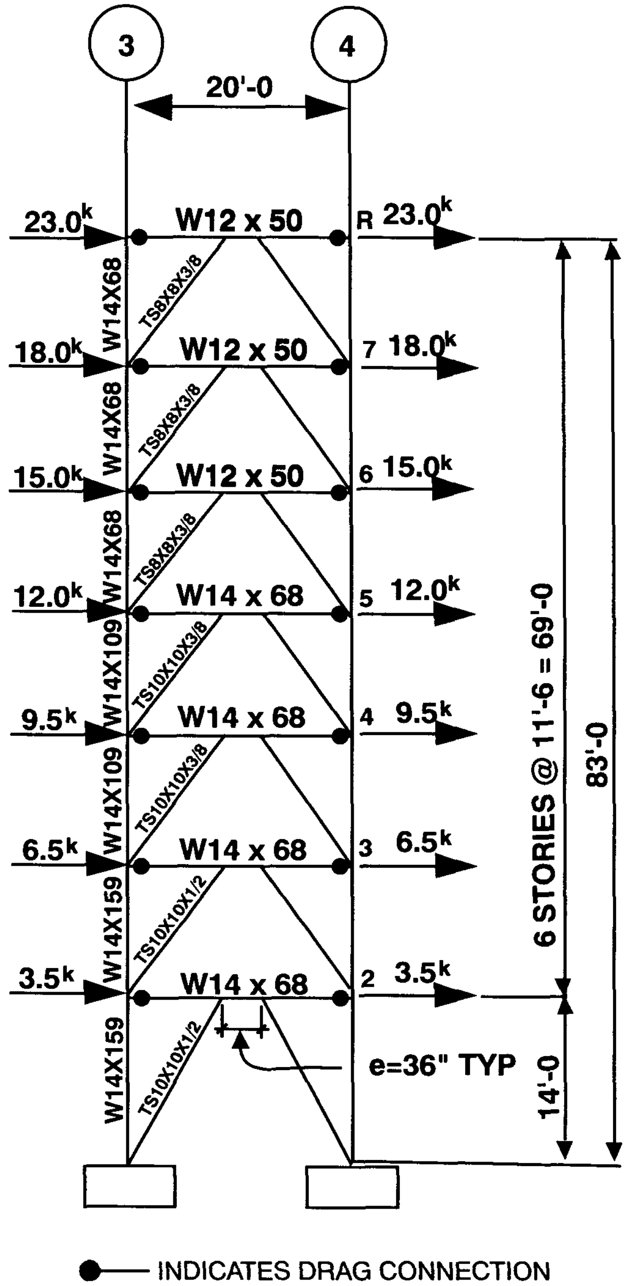
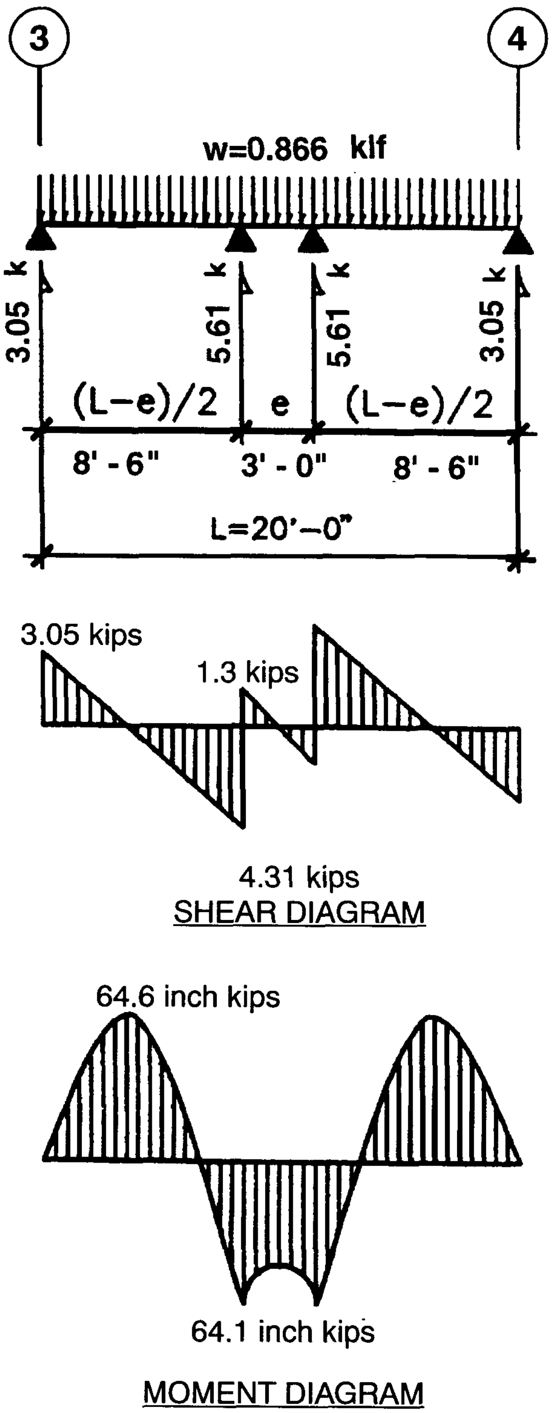


Figure 4
EBF Elevation and Lateral Loads



W14 x 68 $\quad A = 20.0$ in.^{2}

$e = 36$ inches $\quad I = 723.0$ in.^{4}

Figure 5
Beam Gravity Loads

3.3 Column Gravity Loads

Frame columns must be designed to support the critical combination of dead, live, wind and seismic forces. The gravity load tributary to each column can be tabulated for use in the column design. However the column forces due to seismic loads will depend on the strength of the EBF link and cannot be identified until a specific link length and section are chosen.

Table 4 summarizes the gravity loads associated with the vertical frame members for EBFs on grids A & D as shown on Figure 4.

For gravity loading, assume cladding is vertically supported at each level.

TABLE 4
Gravity Column Loads for EBFs on A and D

Level	Trib. Area sq. ft.	∑ Trib. Area sq. ft.	(1) %R	Floor DL kips	Cladding DL kips	LL kips	∑D kips	(8) ∑L kips	∑(D+L) kips
R	(3) 250			67 psf 16.8	(4) 2.6	20 psf 5.0			
7	250	250	0.92	85 psf 21.2	(5) 3.5	50 psf 12.5	19.4	(6) 0.0	19.4
6	250	500	0.92	21.2	3.5	12.5	44.1	12.5	55.6
5	250	750	0.72	21.2	3.5	12.5	68.8	25.0	86.8
4	250	1,000	0.52	21.2	3.5	12.5	93.5	37.5	113.0
3	250	1,250	0.40	21.2	3.5	12.5	118.2	50.0	138.2
2	250	1,500	0.40	21.2	(7) 3.8	12.5	142.9	62.5	167.9
1		1,750	0.40				167.9	75.0	197.9

(1) Reduction factor equal to 1.0 minus (R/100) where R is defined by UBC 1606

(2) Live load reduced by %R

(3) $20\left(\frac{22.5}{2} + 1.25\right)$

(4) 15 psf x 20 (3 + 11.5/2)

(5) 15 psf x 20 (11.5)

(6) Roof live load does not need to be combined with seismic load, UBC 1631.1

(7) 15 psf x 20(11.5 + 14.0)/2

(8) Floor live load not reduced

3.4 Elastic Analysis of Frame

An elastic analysis of the EBFs' lateral deflection is necessary to check for conformance with drift limits, link beam rotation limits and to estimate the building period by Method B. The analysis must account for deflection caused by flexural rotation of the frame and by axial deformation of the columns and braces. Elastic shear deformation of the beams and links should also be included. Most designers use a 2-D elastic plane frame computer analysis. The effect of shear deformation on the frame displacement depends on the size and the length of beams and links. See Table 4A for effect of shear deformation in this example which has member sizes as shown in Figure 4.

Level	With Shear Deformation		Without Shear Deformation		Ratio of δ_x' to δ_x
	Total δ_i in.	Story δ_x in.	Total δ_x' in.	Story δ_x' in.	
R	1.978	0.254	1.713	0.238	0.94
7	1.724	0.309	1.476	0.281	0.91
6	1.415	0.337	1.195	0.301	0.89
5	1.078	0.293	0.894	0.259	0.88
4	0.785	0.276	0.636	0.237	0.86
3	0.509	0.238	0.399	0.196	0.82
2	0.271	0.271	0.203	0.203	0.75

TABLE 4A
Effect of Shear Deformation On Frame Displacement

TABLE 5
Elastic Analysis Summary

Level	V_x kips	Total δ_i in.	Story δ_x in.	(1) P_{LINK} kips	(2) P_{BEAM} kips	(3) V_{LINK} kips	(3) M_{LINK} in. kips	(4) V_{LINK} kips	(4) M_{LINK} in. kips	SIZE & e lb./ft. & inches
R	46	1.978	0.254	0	24.6	28	496	0.8	41	12 x 50 *e* = 36
7	82	1.724	0.309	0	42.3	47	846	1.3	64	12 x 50 *e* = 36
6	112	1.415	0.337	0	55.9	62	1,122	1.3	64	12 x 50 *e* = 36
5	136	1.078	0.293	0	69.9	79	1,426	1.3	64	14 x 68 *e* = 36
4	155	0.785	0.276	0	78.3	88	1,586	1.3	64	14 x 68 *e* = 36
3	168	0.509	0.238	0	87.7	98	1,768	1.3	64	14 x 68 *e* = 36
2	175	0.271	0.271	0	90.1	122	2,196	1.3	64	14 x 68 *e* = 36

(1) P_{LINK} = 0 when equal lateral loads are applied on both sides of the frame.
(2) Axial load due to applied lateral load.
(3) Link reactions due to applied lateral load.
(4) Link reactions due to applied vertical load.

Table 5 summarizes the results of a 2-D elastic plane frame from computer analysis for the configuration shown in Figure 4. For this example, the lateral load shown in Figure 4 was equally applied to both sides of the frame. The tabulated axial load in the link, P_{LINK} and the tabulated axial load in the beam, $P_{BEAM.}$ reflect this distribution. The beam gravity shear, V_{VERT} and bending moment, M_{VERT} are included in the table although they were not included in the deflection analysis.

TABLE 6
Values Used To Determine The Building Period

Level	w_i kips	f_i kips	δ_i in.	$w_i\delta_i^2$	$f_i\delta_i$
R	687	90	1.978	2,688	178.0
7	874	70	1.724	2,598	120.7
6	874	59	1.415	1,750	83.5
5	874	47	1.078	1,016	50.7
4	874	36	0.785	539	28.3
3	874	25	0.509	226	12.7
2	874	13	0.271	64	3.5
Σ	5,931	340		8,881	477.4

The results of the elastic analysis can be used to estimate the building period using Method B. See Table 6.

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \Big/ \left(g\sum_{i=1}^{n} f_i\delta_i\right)} \qquad \text{UBC(28-5)}$$

$$T=2\pi\sqrt{8{,}881/(386.*477.4)} = 1.38 \text{ seconds}$$

Note T≥ 1.073 seconds which confirm the assumption that $T_{METHOD\ B} = 1.3\ T_{METHOD\ A}$ was valid for the stress design of this frame. If deflection (drift) governs the design, UBC 1628.8.3 allows the base shear to be reduced by using the building period determined above where T = 1.38 seconds.

3.5 Deflection Check of Frame

UBC 1628.8.2 limits the elastic story drift under design lateral loads. For buildings having a period over 0.7 seconds:

$$\delta_x < \frac{0.03h}{R_w} = 0.003h < 0.004h \qquad \text{UBC 1628.8.2}$$

Checking the deflection for the second floor relative to the first floor:

$$\delta_{MAX} < \frac{0.03h}{R_W} = 0.03\,\frac{(14)(12)}{10} = 0.504 \text{ in.} > 0.271 \text{ in.}$$

Checking the deflection for the upper stories:

$$\delta_{MAX} < \frac{0.03\,h}{R_W} = 0.03\,\frac{(11.5)(12)}{10} = 0.414 \text{ in.} > 0.337 \text{ in.}$$

Thus, per Table 5 all floors are o.k.

3.6 Link Size

As shown in Figure 6, the link design shear from lateral load can be determined independently of the link length, bracing configuration, section properties or the elastic analysis shown in Table 5.

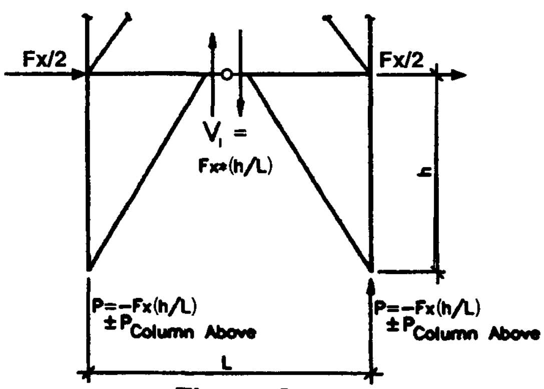


Figure 6. Link Shear From Lateral Load

The unfactored seismic design loads for the EBF on Grid A at the 2nd level are shown in Figure 7.

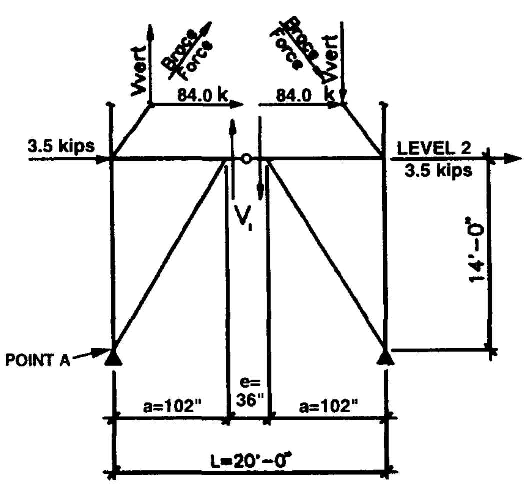


Figure 7. Unfactored Seismic Design Loads

The link design shear from gravity load is usually not significant. The gravity shear is shown in Figure 5 and is included in this example. Taking moments about Point A of Figure 7, it can be shows that:

$$V_l = F_x\left(\frac{h}{L}\right) + V_{VERT} = (7 + 84 + 84)\left(\frac{14}{20}\right) + 1.3$$

$= 123.8$ kips

This corresponds very closely to the elastic analysis per Table 5.

UBC 2211.10.5 limits the web shear to 0.8 V_s
The requirement that the link beam web shear not exceed .80 of the shear strength is a conversion to an allowable stress approach for the design of link (ref. 11, p. 332 C709.5).

$$V_l \le 0.80V_s = 0.80(0.55)F_y dt_w \qquad \text{UBC 2211.10.5, UBC 2211.4.2}$$

$F_y = 50$ ksi

$$dt_{w\,min} = \frac{123.8}{0.80(0.55)(50)} = 5.63 \text{ in.}^2$$

The most efficient link section usually:

1) Optimizes the required shear area (min dt_w);

2) Is the deepest possible while complying with the compact web criteria (max $\frac{d}{t_w}$)

3) Has compact flanges with sufficient bending capacity to ensure shear failure of the section under ultimate load (recommended e max = 1.3 $\frac{M_s}{V_s}$)

<u>Try W 14 x 68</u>

r_x = 6.01 in.	dt_w = 5.83 in.^{2}
r_y = 2.46 in.	b_f = 10.035 in.
A = 20.0 in.^{2}	A_f = 7.23 in.^{2}
S_x = 103.0 in.^{3}	
Z_x = 115.0 in.^{3}	Z_f = 96.3 in.^{3}

Notes:
1) The provided dt_W is only 5% greater than the minimum required in this example. Thus, it is a very efficient design. The section was chosen in order that the axial and flexural requirements discussed in Sections 3.12, "Combined Link Loads" and 3.15, "Beam Analysis," are also satisfied, as well as the requirements for compact web and flanges.

2) The adverse consequences of any excess shear capacity are shown in Sections 3.12, "Combined Link Loads"; 3.15, "Beam Analysis"; 3.17, "Brace Analysis"; 3.18, "Column Analysis"; 3.19, "Foundation Design"; 3.21, "Beam Lateral Buckling"; 3.22, "Brace to Beam Connection"; and 3.23, "Brace to Column and Beam Connection".

The web compactness criterion is dependent on the axial stress in the section which is unknown until a trial selection is made. Built-up sections can be fabricated to optimize the link beam section properties. **Excess capacity in the link can be costly as other elements of the frame are sized to ensure that the link is the weakest portion of the frame.**

3.7 Link Shear Strength and Link Strength Factor

In order to assure that the link is the only inelastic mechanism in an EBF, all components outside the link are designed to have a strength greater than the link. If excess link capacity is provided, the strength of all other parts of the EBF must also be increased.

$V_s = 0.55F_y dt_w = 0.55(50)(5.83) = 160.3$ kips.

For strength checks all prescribed code loads will be increased by ϕ, the link strength factor. This design load shall be used for determining strength requirements for other elements of the EBF.

$$\phi = \frac{V_s}{V_i} = \frac{160.3}{123.8} = 1.29$$

From UBC 2211.10.5, $\phi_{MIN} = 1/0.80 = 1.25$. Thus, the selection of the W14x68 is a very efficient design.

The UBC does not require drag struts, diaphragms or other lateral components beyond the EBF to be designed for loads in excess of those attributed to these components in the lateral analysis. SEAOC recommends that collectors directly connecting to the EBF be designed to provide sufficient strength to deliver the forces corresponding to link beam yield (ref .11, p. 335 C709.17 & C709.19.

Recognizing that the lateral system has been selected and analyzed on the presumption that yielding of the link will be the method of energy dissipation, the author recommends that the strength capacity of drag struts, diaphragms and other lateral components exceed the yield strength of the link.

3.8 Beam Compact Flange

Check compact flange criterion:

$$\frac{b_f}{2t_f} = \frac{10.035}{2(0.72)} = 6.97 < \frac{52}{\sqrt{F_y}} = 7.36 \qquad \text{UBC 2211.10.2}$$

To meet this requirement, it sometimes may be prudent to use a section built up from plate elements in order to prevent local buckling.

3.9 Link Length

The influence of link length on the behavior of EBFs is discussed in the introduction, Section 1.4. To assure shear ductility, the link length will be limited to $1.3M_s/V_s$.

$V_s = 0.55F_y dt_w = 0.55(50)(5.83)$ UBC 2211.4.2

$= 160.3$ kips

$M_s = Z_x F_y = 115\ (50) = 5{,}750$ in. kips

$$e = 1.3\frac{M_s}{V_s} = 1.3\left(\frac{5750}{160.3}\right) = 46.6 \text{ in.}$$

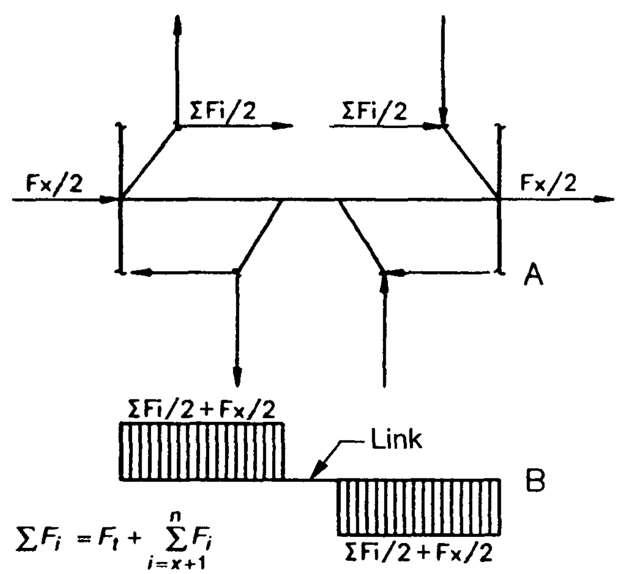
A W14 X 68 with e = 0.15L = 0.15 (20 x 12) = 36" per Section 1.4 will be a shear link unless the axial force in the link is very large.

3.10 Beam and Link Axial Loads

In an EBF link, the axial force may reduce the flexural link capacity. The link should be checked for the effect of axial forces combined with bending forces. This combination could produce flange yielding before web shear yielding.

To account for axial load in a link beam requires an understanding of how the lateral forces travel through the diaphragm, into the beam and into the braces. The arrangement of braces and the direction from which lateral loads are applied can modify the axial force distribution in the link beams. Caution should be used when taking these forces from a computer model. Most computer programs which use rigid diaphragm assumptions do not model the axial force distribution in the beams.

In a symmetrical chevron configuration EBF centered on the building grid, symmetric drag struts would typically collect the lateral loads as shown in Figure 8. In Figure 8, $\sum F_i$ is the sum of the lateral forces above the frame being considered. F_x is the lateral force from the story being considered.



$$\sum F_i = F_t + \sum_{i=x+1}^{n} F_i$$

Figure 8
Beam Axial Loads

The EBFs on grids A and D are located in the center of the building. This example will assume that the drag struts occur on both sides of the frame and that the lateral force is applied to both sides of a symmetrically braced frame as shown in Figure 8. For the 2nd floor beam, $\sum F_i$=168 kips and F_x = 7 kips per the lateral load distribution shown in Table 2. $F_x/2$ represents the minimum diaphragm drag force. In this case the diaphragm design forces shown in Table 3 are greater than the distributed lateral forces. F_x will be governed by the minimum allowable diaphragm design load per Table 3.

$F_x = 123/2 = 61.5$ kips   2 EBFs per story

$F_x/2 = 30.8$ kips > 3.5 kips   Use 30.8 kips

As shown in Figure 8B, the axial load in the link=0.

3.11 Beam Compact Web

The maximum $\frac{d}{t_w}$ ratio permitted for compact beam sections is dependent on the axial load in the beam. Sections noted F_y''' in the AISC manual (ref.12) have compact webs for all combinations of axial stress when the yield strength is less than the tabulated values.

If a beam section is chosen that does not have a compact web for all axial loads, the section should be checked using allowable stress design UBC Chapter 22, Division IX, Table B5.1. The web should be compact along the full length of the beam.

For the second level W14 x 68:

$$\frac{d}{t_w} = \frac{14.04}{0.415} = 33.8$$

$$A = 20.0 \text{ in}^2$$

$$f_a = \frac{\Sigma F_i/2 + F_x/2}{A} = \frac{84.0 + 30.8}{20.0} = 5.74 \text{ ksi}$$

$$\frac{f_a}{F_y} = \frac{5.74}{50.0} = 0.11 < 0.16$$

$$\left(\frac{d}{t_w}\right)_{MAX} = \frac{640}{\sqrt{F_y}}\left(1 - 3.74\frac{f_a}{F_y}\right)$$

$$= \frac{640}{\sqrt{50}}[1 - 3.74(0.11)] = 53.3 > 33.8 \text{ o.k.}$$

UBC 2211.10.5 does not allow doubler plates to reduce $\frac{d}{t_w}$ requirements for a link beam.

3.12 Combined Link Loads

The design of a shear link is based on having sufficient flexural strength to ensure shear failure under ultimate load. Axial force in a link reduces the moment capacity of the section. Consequently, the link needs to be checked for the possibility of the axial force reducing the moment capacity and shifting the first yield from shear to flexure.

UBC 2211.10.3 requires that "where link beam strength is governed by shear, the flexural and axial capacities within the link shall be calculated using the beam flanges only." The SEAOC commentary (ref. 11, p. 330 C709.3) identities links with e < 2.0 $\frac{M_s}{V_s}$ as being governed by shear and subject to this requirement.

The second level link section will be checked using this criteria.

$$P = P_{lu} = \phi P_l = 1.29(0) = 0 \text{ kips}$$

$$M = M_{lu} = \frac{\phi V_l e}{2} = \frac{1.29(123.8)36}{2} = 2{,}875 \text{ in. kips}$$

W14 x 68

$F_y = 50.0$ ksi

$A_f = b_f t_f$

$= (10.035)(0.72)$

$= 7.23$ in.^{2}

$Z_f = (d - t_f)b_f t_f$

$= (14.04\text{-}0.72)(10.035)(0.72)$

$= 96.3$ in.^{3}

$$\frac{P}{2A_f} + \frac{M}{Z_f} = \frac{0}{2(7.23)} + \frac{2875}{96.3} = 29.9 \text{ ksi} \le F_y \text{ o. k.}$$

This provision of the UBC dedicates the web to shear loads and the flanges to axial and flexural loads. This simplifies the analysis of the link. The intent of this provision is to ensure adequate flexural strength at full shear yielding of the link.

Failure to meet this criteria would indicate that flexural yielding could occur before shear yielding and that an alternate section with greater flexural capacity should be selected to provide a shear link.

3.13 Verification of Link Shear Strength and Strength Factor

Returning to the UBC, the strength of the link is used to establish the minimum strength required of elements outside the link. The link shear strength, V_s, was determined in Section 3.7, "Link Shear Strength and Link Strength Factor." The shear in the link when the section has reached flexural capacity may be less then the shear strength of the section. If this is true, the flexure capacity of the section will limit the shear capacity of the link. UBC 2211.10.3 requires that the flexural capacity of the section reduced for axial stress be considered as a possible upper limit of the link capacity when the link beam strength is governed by shear.

$V_s = 0.55F_y d t_w = 160.3$ kips UBC 2211.4.2

$M_{rs} = Z_x(F_y - f_a) = Z_x F_y$ (since $f_a = 0$)

$= 115(50) = 5{,}750$ in. kips UBC 2211.10.3

M_{rs} may limit the shear capacity of the link.

$$V_{rs} = \frac{2M_{rs}}{e} = \frac{2(5{,}750)}{36} = 319.4 \text{ kips}$$

$V_{CONTROLLING\ CAPACITY} = \min(V_s, V_{rs})$

$= \min(160.3, 319.4) = 160.3$ kips

Link strength factor $\phi = \dfrac{V_s}{V_l} = \dfrac{160.3}{123.8} = 1.29$

The shear capacity of this section is governed by the shear strength of the web. It is not governed by the shear which can be developed by the section reduced for the axial load in the link acting in flexure over the length of the link. Thus the link strength factor ϕ has been verified to be = 1.29 per Section 3.7.

3.14 Beam Brace Spacing

UBC 2211.10.18 requires braces to top and bottom flanges at the ends of the link beams. Braces may be required beyond the link. If additional bracing is required, it should be located to optimize the reduction in axial buckling length of the beam. Check to see if braces are required outside the link.

$\ell_{UNBRACED\ MAX} = \dfrac{76b_f}{\sqrt{F_y}} = 108$ in. UBC 2211.10.18

The length of the beam outside the link is 102 inches. No additional bracing is required. (Additional bracing is required for W12 x 50 beams at the 6th, 7th and roof levels.)

3.15 Beam Analysis

Beyond the link, the beam must have sufficient capacity to resist 1.5 times the combined axial and flexural loads corresponding to the link beam strength per UBC 2211.10.13. For axial load is the beam for code seismic loads, see Section 3.10.

$P_{bu} = 1.5\,\phi\,(\Sigma F_i/2 + F_x/2) = 1.5(1.29)(84.0+30.8) = 222$ kips

$$M_{bu} = \frac{1.5\,\phi V_l e}{2} = \frac{1.5 V_s e}{2} = \frac{1.5(160.3)(36)}{2} = 4{,}328 \text{ in. kips}$$

The beam design moment beyond the link may be determined from an elastic analysis of the frame. If this is done:

$M_{bu} = 1.5\phi M_{be}$ where M_{be} is from an elastic analysis. Although not significant in this example, the beam gravity moment, as shown in Figure 5, may be included in M_{be}.

Check the 102 inch unbraced beam segment outside of the link using the plastic design criteria (UBC Chapter 22, Division IX, Chapter N, ref. 2).

For W14 x 68:

$$\frac{kl}{r_y} = \frac{102}{2.46} = 41.5$$

$F_{ay} = 25.55$ ksi

$F'_{ey} = 84.65$ ksi

$$\frac{kl}{r_x} = \frac{102}{6.01} = 17$$

$F_{ax} = 28.61$ ksi

$F'_{ex} = 517$ ksi

$P_{cr} = 1.7F_a A = 1.7(25.55)(20.0) = 868.7$ kips

$P_e = \dfrac{23}{12} F'_e A = \dfrac{23}{12}(84.65)(20.0) = 3{,}245$ kips

$P_y = F_y A = 50\,(20) = 1{,}000$ kips

$M_m = M_p = F_y Z_x = (50)(115) = 5{,}750$ in. kips

$C_m = 0.85$

$$\frac{P}{P_{cr}} + \frac{C_m M}{\left(1 - \frac{P}{P_e}\right) M_m} = \frac{222}{868.7} + \frac{0.85(4{,}328)}{\left(1 - \frac{222}{3{,}245}\right) 5{,}750}$$

$= 0.94 < 1.0$ ∴ o.k. UBC (N4-2)

$$\frac{P}{P_y} + \frac{M}{1.18M_p} = \frac{222}{1{,}000} + \frac{4{,}328}{1.18(5{,}750)} = 0.22 + 0.64$$

$= 0.86 < 1.0$ ∴ o.k. UBC (N4-3)

W14x68 o.k.

The beam typically carries large axial load. This tends to buckle the beam in a non-ductile manner. The presence of a concrete slab provides a significant stabilizing contribution to the beam. Conservative design of the beam, particularly in elevator cores or other locations where a slab or other bracing is restricted, is advised.

3.16 Link Rotation

Ductile behavior of an EBF requires inelastic deformation of the link. This deformation causes the link to rotate. UBC 2211.10.4 imposes upper bounds on the link rotation to limit the ductility demand on the frame.

To estimate the EBFs' deformation during a major seismic event, the elastic deflections resulting from the applied code lateral loads are factored up by $\frac{3R_w}{8}$.

Under this extreme load, plastic hinges are assumed to have formed in the link. Consequently, the EBF may be modeled as a rigid body with pivot points at the link and an imposed deformation. The link rotation can be determined from the lateral deflection and the frame geometry.

Consider the general chevron configuration EBF shown in Figure 9A, where θ = rotation of the link realtive to the rest of the beam.

$$\theta_1 = \frac{\delta}{h_1} \qquad \theta_2 = \frac{\delta}{h_2}$$

$$\delta_1 = \theta_1 a_1 \qquad \delta_2 = \theta_2 a_2$$

$$\theta = \theta_1 + \frac{\delta_1}{e} + \frac{\delta_2}{e}$$

$$\theta = \frac{\delta_1}{h_1} + \frac{\delta a_1}{h_1 e} + \frac{\delta a_2}{h_2 e}$$

For the symmetric chevron configuration shown in Figure 9B:

$$a_1 = a_2 \qquad h_1 = h_2$$

$$\theta = \frac{\delta}{h}\left(1 + \frac{2a}{e}\right)$$

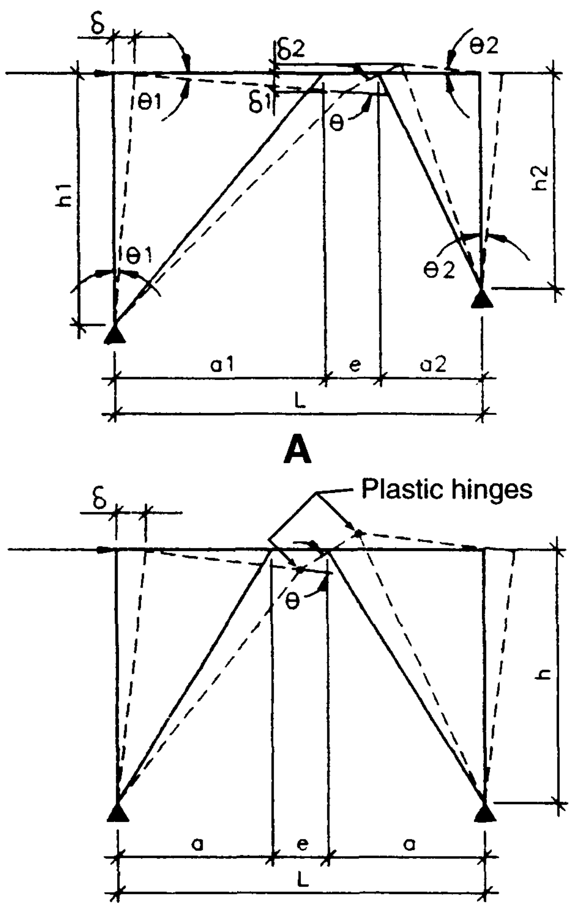


Figure 9
Link Rotation

For the second level frame:

δ = design drift for the EBF.

$$\delta = \frac{3R_w}{8}\delta_x$$

δ_x = elastic deformation due to the seismic design load.

δ_x = 0.271 from the elastic analysis of this frame (Table 5).

$$\delta = \frac{3(10)}{(8)}(0.271) = 1.016 \text{ in.}$$

$$\theta = \frac{\delta}{h}\left(1 + \frac{2a}{e}\right) = \frac{1.016}{14(12)}\left(1 + \frac{2(102)}{36}\right) = 0.0403 \text{ radians}$$

$\theta = 0.0403 < \theta_{MAX} = 0.060$ radians $\therefore$ o.k. UBC 2211.10.4.1

The maximum allowable link rotation can also be used to determine the minimum allowable link length.

$$\theta_{MAX} = \frac{\delta}{h}\left(1 + \frac{2a}{e_{MIN}}\right) = \frac{\delta}{h}\left(\frac{L}{e_{MIN}}\right)$$

$$e_{MIN} = \frac{\delta}{h}\frac{L}{\theta_{MAX}} = \left(\frac{3R_w}{8}\frac{\delta_x}{h}\right)\left(\frac{L}{\theta_{MAX}}\right) = \frac{3(10)}{(8)}\frac{(0.271)}{(14)}\frac{(20)}{(0.060)} = 24.2 \text{ in.}$$

As noted in the introduction, longer links will reduce damage to the floor structure. However, longer links will result in increased drift under lateral load.

3.17 Brace Analysis

To ensure that the strength of the brace exceeds the link strength, UBC 2211.10.13 requires "each brace to have a compressive strength of at least 1.5 times the axial force corresponding to the controlling link beam strength." The link beam strength is determined from V_s or V_{rs}. In this frame V_s is smaller and governs the brace design as shown in Section 3.13.

The brace design force can be determined knowing V_s and the frame geometry as shown in Figure 10:

V_{br}, the beam shear force to be resisted by the brace, includes a component from both the link and from the beam outside the link as shown in Figure 11.

$$V_{br} = 1.5(V_b + V_s)$$

$$V_s = 160.3 \text{ kips}$$

$$M_s = \frac{V_s e}{2} = \frac{160.3(36)}{2} = 2{,}885 \text{ in. kips}$$

$$V_b = \frac{M_s}{(L - e)/2} = \frac{2{,}885}{(240 - 36)/2} = 28.3 \text{ kips}$$

$$V_{br} = 1.5(28.3 + 160.3) = 283 \text{ kips}$$

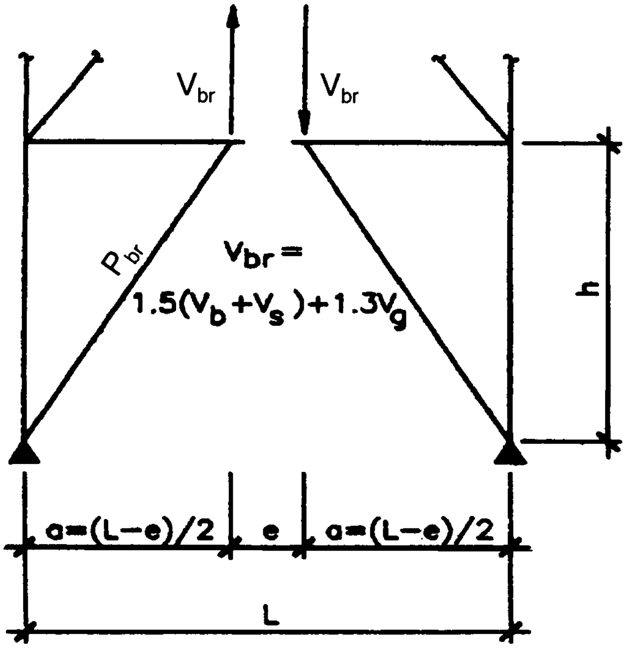


Figure 10
Brace Shear Force

The braces support part of the beam gravity load. Although the gravity load is a small portion of the brace design load, it is included in this example.

V_g = shear from beam gravity loading (see Figure 5).

LF = 1.3 = plastic design load factor UBC Chapter 22, Division IX, Section N1

$$L_{br} = \sqrt{a^2 + h^2} = \sqrt{8.5^2 + 14^2} = 16.38 \text{ feet}$$

$$P_{br} = V_{br}\left(\frac{L_{br}}{h}\right) + V_g\left(\frac{L_{br}}{h}\right)LF$$

$$= 283\left(\frac{16.38}{14}\right) + 5.61\left(\frac{16.38}{14}\right)1.3$$

$$= 331.1 + 8.5 = 339.6 \text{ kips}$$

The UBC does not require a moment connection between the brace and the beam. If an analysis of the frame is done assuming that this is a pinned connection which includes gravity loads, the most critical bending moment to unbraced length combination may occur outside the link. This could be contrary to the design strategy of concentrating the critical stresses in the link. In practice, the connection between the brace and the link is typically capable of transferring moment from the beam to the brace (ref. 9, p. 500). This capacity is advantageous in keeping the critical stress location within the link.

If a computer analysis is used to model the frame, this connection should be assumed fixed. For preliminary sizing it is reasonable to assume that the brace is pinned and increase the design axial load 15-20% to account for the bending effects.

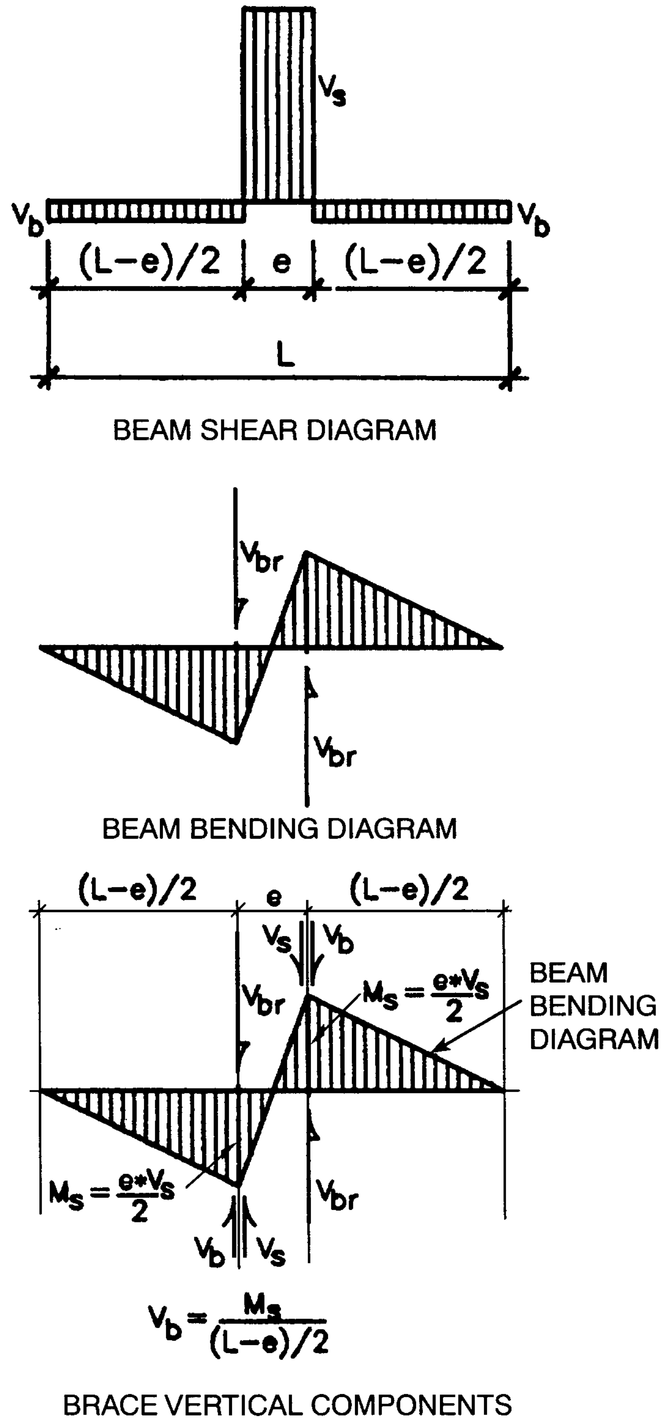


Figure 11
Brace Vertical Force Components

The moment distribution resulting from an elastic computer analysis of the frame in this example is shown in Figure 12.

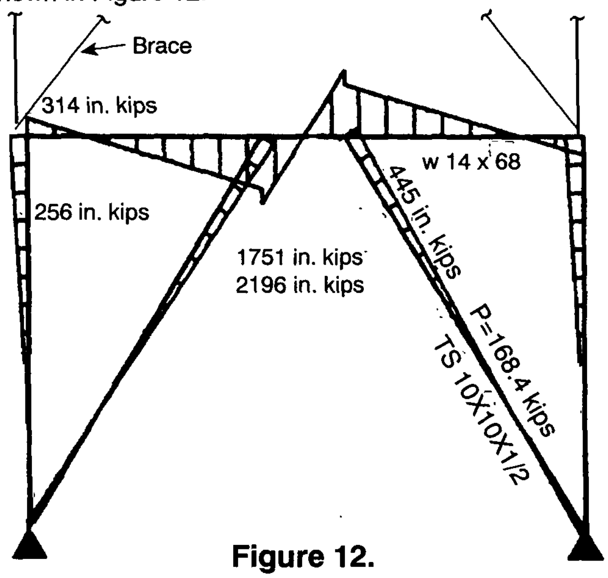


Figure 12.
Moment Distribution Between the Beam and Brace

For this example the elastic computer analysis compression and moment on the brace will be used and scaled by ϕ, the link strength factor, to keep the brace design consistent with capacity sizing.

The elastic computer analysis includes the effect of fixity between the beam and the column. It also accounts for the fixity between the beam and the brace. The elastic computer analysis did not include the gravity loading. The factored compression in the brace, due to lateral load, by the hand analysis is 331 kips. This compares very well with the factored compression brace load of 326 kips from the computer analysis.

$P = 1.5\phi P_{ELASTIC} = 1.5(1.29)(168.4) = 326$ kips
$M = 1.5\phi M_{ELASTIC} = 1.5(1.29)(445) = 861$ in. kips

Check the *TS*10x10x1/2 (Use plastic design criteria)

$F_y = 46$ ksi
$A = 18.4$ in.^{2}
$Z = 64.6$ in.^{3}
$r = 3.84$ in.

$$\frac{kl}{r} = \frac{16.38(12)}{3.84} = 51$$

$F_a = 22.6$ ksi

$F_e' = 57.9$ ksi

$P_{cr} = 1.7F_aA = 1.7(22.6)(18.4) = 707$ kips

$$P_e = \left(\frac{23}{12}\right)F_e'A = \left(\frac{23}{12}\right)(57.9)(18.4)$$

$= 2{,}042$ kips

$P_y = F_yA = 46(18.4) = 846$ kips

$M_m = M_p = F_yZ = 46(64.6) = 2{,}972$ in. kips

$C_m = 0.85$

$$\frac{P}{P_{cr}} + \frac{C_mM}{\left(1-\frac{P}{P_e}\right)M_m} = \frac{326}{707} + \frac{0.85(861)}{\left(1-\frac{320}{2{,}042}\right)2{,}972}$$

$= 0.75 < 1.0$ ... o.k. UBC (N4-2)

$$\frac{P}{P_y} + \frac{M}{1.18M_p} = \frac{326}{846} + \frac{861}{1.18(2{,}972)}$$

$= 0.63 < 1.0$ ... o.k. UBC (N4-3)

TS 10x10x½ o.k. (Could be reduced)

3.18 Column Analysis

UBC 2211.10.14 requires columns to remain elastic with all of the EBF links in a bay at 1.25 times their strength. Each link beam strength, should be determined from V_s or V_{rs} as appropriate. In this example, V_s governs as shown in Section 3.13.

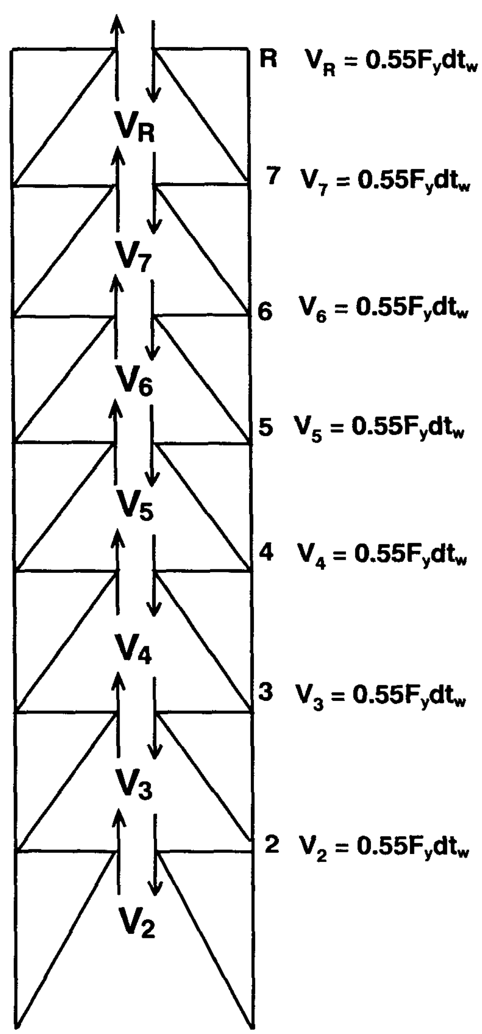


Figure 13
Shear Capacity of the Links

The controlling link strengths for the column design are shown in Table 7.

TABLE 7
Controlling Link Strengths

Level	Link Size	dt_w in.^{2}	V_s(1) kips	ΣV_s kips
R	W12x50	4.51	124	124
7	W12x50	4.51	124	248
6	W12x50	4.51	124	372
5	W14x68	5.83	160	532
4	W14x68	5.83	160	692
3	W14x68	5.83	160	852
2	W14x68	5.83	160	1,012

(1) $V_s = 0.55\ F_y dt_w$

$$P_{cu} = 1.25\left[\sum_{i=x}^{n}\min\ (V_s,\ V_{rs})\right] + 1.3(P_{dl} + P_{ll})$$

For the first level column:

$$\sum_{i=2}^{R}\min\ (V_s,\ V_{rs}) = 1{,}012 \qquad \text{Table 7}$$

ΣD = 168 kips — Table 4

$\Sigma\ (D+L)$ = 198 kips — Table 4

$P_{cu} = 1.25(1{,}012) + 1.3(198) = 1{,}522$ kips

In this frame the beam to column and brace to column connections could be designed as pins per UBC 2211.10.19. If they are designed as fixed, the elastic column moments should be scaled up and included in the column design. As shown in Figure 12, they were modeled as fixed. The moment in the column will be included in this example.

$M_{cu} = 1.25\ \ \phi\ M_{ce}$

M_{cu} = ultimate design moment in the column

ϕ = link strength factor

M_{ce} = moment in the column from an elastic analysis of the design seismic forces

$M_{CU} = 1.25\ (1.29 \times 256) = 413$ in. kips

The column is oriented for strong axis bending of the EBF.

If the column is subjected to minor axis bending, from girders or other asymmetric loads, the minor axis bending must be included in the combined compression and bending interaction checks. Minor axis bending has been omitted in this example.

Check the W14x159

Use plastic design criteria per UBC Chapter 22, Division IX, Chapter N.

F_y = 50 ksi

A = 46.7 in.^{2}

Z_y = 287 in.^{3}

r_y = 4.00 in.

$$\frac{kl}{r_y} = \frac{1.0(14)(12)}{4.00} = 42$$

Note: k = 1.0 is conservative for columns braced against translation with some degree of rotational restraint provided by the foundation anchorage and the second floor beams. Although the stiffness of a shear link EBF is slightly less than a CBF, k= 1.0 is a reasonable assumption for most EBF frames.

$F_a = 25.55$ ksi

$F_e' = 84.65$ ksi

$P_{cr} = 1.7F_aA = 1.7(25.55)(46.7) = 2{,}028$ kips

$$P_e = \left(\frac{23}{12}\right) F_e'A = \left(\frac{23}{12}\right)(84.65)(46.7)$$

$= 7{,}577$ kips

$P_y = F_yA = 50(46.7) = 2{,}335$ kips

$M_m = M_p = F_yZ = 50(287) = 14{,}350$ in. kips

$C_m = 0.85$

$$\frac{P}{P_{cr}} + \frac{C_mM}{\left(1 - \frac{P}{P_e}\right)M_m} = \frac{1{,}522}{2{,}028} + \frac{0.85(413)}{\left(1 - \frac{1{,}522}{7{,}577}\right)14{,}350}$$

$= 0.78 < 1.0\ \therefore$ o.k. UBC (N4-2)

$$\frac{P}{P_y} + \frac{M}{1.18M_p} = \frac{1{,}522}{2{,}335} + \frac{413}{1.18(14{,}350)}$$

$= 0.68 < 1.0$ ... o.k. UBC (N4-2)

W14x159 o.k. (Could be reduced)

The intention of UBC 2211.10.14 is to ensure that the columns do not fail prior to the full utilization of the energy dissipation capacity of the link. Consequently, it a link is designed with more capacity than required, all of the columns below the link will need to have a corresponding excess capacity. UBC 2211.5.1 provides an upper limit to the column strength requirement. Columns may be designed for a maximum compression or the lesser of:

$$P_{MAX} = 1.25\left[\sum_{i=x}^{n}\min(V_s\,,\,V_{rs})\right] + 1.3\,(P_{dl} + P_{ll})$$

or

$$P_{MAX} = \left(\frac{3R_w}{8}\right)P_E + 1.0P_{dl} + 0.7P_{ll}$$

and for a maximum uplift of the lesser of:

$$P_{MIN} = 1.25\left[\sum_{i=x}^{n}\min(V_s\,,\,V_{rs})\right] - 0.85P_{dl}$$

or

$$P_{MIN} = \left(\frac{3R_w}{8}\right)P_E - 0.85P_{dl}$$

3.19 Foundation Design

The design of the foundation requires a review of the structural objective of the foundation. Designers should consider the ductility of the foundation in relation to the ductility of the superstructure. Brittle foundation systems should be designed to higher loads than ductile or flexible foundations. The foundation design forces should consider the capacity of the superstructure to transmit force. Two approaches to the foundation design will be presented

The first and most prevalent approach is to design the foundations for the code required dead load, live load, seismic overturning and seismic sliding forces. When this approach is followed, it is probable that the reactions from the columns into the foundation will be significantly less than the column capacities. UBC 1809.3 requires the connection of the superstructure elements to the foundation be capable of transmitting the forces for which the elements were designed. If the foundation design is based on less force than the column design, the capacity of the connection between the column and the foundation should exceed the actual foundation capacity. This will ensure that a frame overload would occur in the soil structure interface and not within the confines of the structure. If this approach is followed, the designer must realize that the links may not yield prior to the foundation reaching its design strength. If the links do not yield, the frame will behave like a concentrically braced frame. This behavior is inconsistent with the assumed R_w.

The second approach (although not required by the UBC) is to design the foundation to exceed the capacity of the superstructure. In this approach the design objective is to ensure that any failure of structural components occurs in the ductile frame. The foundations must be capable of transmitting the factored column capacity design loads to the soil. In this approach the foundation must be designed for a maximum compression of the lesser of:

$$P_{MAX} = 1.25\left[\sum_{i=x}^{n}\min(V_s\,,\,V_{rs})\right] + 1.3\,(P_{dl} + P_{ll})$$

or

$$P_{MAX} = \left(\frac{3R_w}{8}\right)P_E + 1.0P_{dl} + 0.7P_{ll}$$

and for a maximum uplift of the lesser of:

$$P_{MIN} = 1.25\left[\sum_{i=x}^{n}\min(V_s\,,\,V_{rs})\right] - 0.85P_{dl}$$

or

$$P_{MIN} = \left(\frac{3R_w}{8}\right)P_E - 0.85P_{dl}$$

If this approach is used, base plates and anchor bolts should be sized such that their strength as defined by UBC 2211.4.2 and 1925.2, respectively, exceeds the maximum combined axial and shear loads. Anchor bolts should be embedded sufficiently to develop their combined shear and tensile strength. Embedding the column in the foundation may be the most practical way to do this for large loads. Concrete elements of the foundation may be designed using the above as ultimate loads with no additional load factors.

A qualified geotechnical engineer should be encouraged to provide ultimate soil capacity design values for use with the above. The allowable soil capacities with a one-third increase should also be checked against the code applied lateral forces per UBC 1809.2.

3.20 Beam Stiffeners

Beam stiffeners are used to prevent buckling of the web and ensure a ductile shear yielding of the web. Stiffeners are required at each end of the link and at regular intervals within the link.

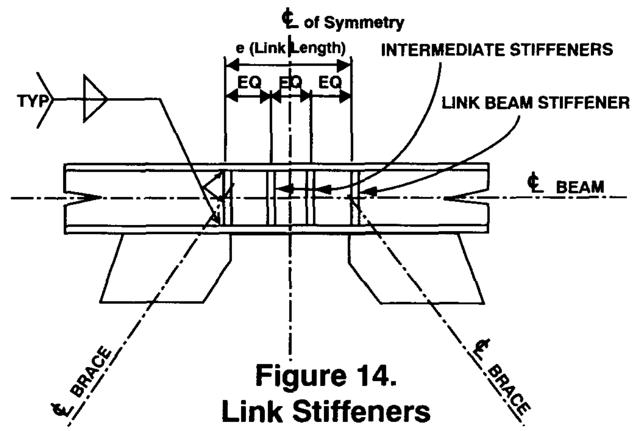


Figure 14. Link Stiffeners

UBC 2211.10.7 requires full depth web stiffeners on both sides of the beam web at the brace end of the link beam.

For the W14 x 68:

Min. combined width $\geq b_f - 2t_w$ UBC 2211.10.7

$$2b > 10.035 - 2(0.415) = 9.2 \text{ in.}$$

Min. thickness $0.75t_w$ or 3/8 in. UBC 2211.10.7

$$t > 0.75(0.415) = 0.31 \text{ in.}$$

Use $4^3/_4$" x $^3/_8$" stiffeners each side.

UBC 2211.10.8 requires intermediate full depth web stiffeners when the beam strength is controlled by V_s or when the shear from M_{rs} exceeds $0.45F_y dt_w$. Therefore, intermediate stiffeners are required for this link. UBC 2211.10.9 identifies the spacing limits as a function of the link rotation. For rotations of less than 0.03 radians, the maximum spacing is $56t_w - \frac{d}{5}$. For link beams with rotation angle of 0.06 radians, the spacing shall not exceed 38 t_w - d/5. Interpolation may be used for rotation angles between 0.03 and 0.06 radians.

$\theta \le 0.03$ radians

$$\text{Maximum Spacing} = 56t_w - \frac{d}{5}$$
$$= 56 \times (0.415) - \frac{14.04}{5}$$
$$= 20.4"$$

$\theta = 0.06$ radians

$$\text{Maximum Spacing} = 38t_w - \frac{d}{5}$$
$$= 38 \times (0.415) - \frac{14.04}{5}$$
$$= 13"$$

$\theta = 0.0403$ radians (See Section 3.16)

$$\text{Maximum Spacing} = 13 + \frac{20.4 - 13}{0.03}(0.0103)$$
$$= 15.5"$$

For a 36" link, two intermediate stiffeners are required as shown in Figure 14.

UBC 2211.10.10 notes that for beams less than 24 inches in depth, intermediate stiffeners are required on only one side of the web.

Min. width > $(b_f/2) - t_w$ UBC 2211.10.10

$b > 10.035/2 - 0.415 = 4.6$ in.
Min. thickness = 3/8 in. UBC 2211.10.10

Use 4³/₄" x ³/₈"stiffener on one side.

The link end and intermediate stiffeners are the same size in this example.

UBC 2211.10.11 requires welds connecting the stiffener to the web to develop $A_{st}F_y$, and welds connecting the stiffener to the flanges to develop $A_{st}F_y/4$.

$A_{st} = 4.75(0.375) = 1.78 \text{ in.}^2$

$A_{st}F_y = 1.78(50) = 8.9$ kips

Weld capacity = 1.7 Allowable UBC 2211.4.2

Use E70 electrodes, SMA fillet welds, Grade 50 base metal.

$F_w = 1.7(0.30)(70)(0.707) = 25.2$ ksi UBC Chapter 22, Division IX, Table J2.5

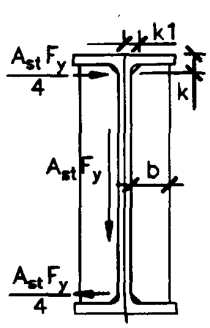


Figure 15
Stiffener Weld Forces

Weld to Web:

Find the minimum weld size, "a," if the full available length of the web is used.

$$a_{WEB,MIN} = \frac{A_{st}F_y}{F_w(d-2k)} \text{ in.}$$

$$= \frac{1.78(50)}{25.2(14.04 - 2(1.5))} = 0.32 \text{ in.}$$

Check the minimum weld size for the base metal thickness.

$t_w = 0.415$ in., $a_{MIN} = 3/16$ in. UBC Table J2.4

Use 3/8" full height fillet weld to beam web.

Weld to Flanges:

$$a_{FLANGE,MIN} = \frac{A_{st}F_y/4}{F_w(b-k_1)}$$

$$= \frac{1.78(50)/4}{25.2(4.75 - 15/16)}$$

$$= 0.23 \text{ in.}$$

Check, the minimum weld for the base metal thickness.

$t_f = 0.72$ in., $a_{MIN} = 1/4$ in. UBC Table J2.4

Use 1/4" fillet weld to beam flange.

3.21 Beam Lateral Bracing

UBC 2211.10.18 requires the top and bottom flanges to be braced at the ends of link beams and at specific intervals. This requirement is independent of the EBF configuration.

The UBC requires the bracing to resist 6.0% of the beam flange strength at the ends of link beam. Thus, for a W14x68 beam:

$$P_{BRACE} = 0.060\, F_Y b_f t_f = 0.060(50)(10.035)(0.72)$$

$$= 21.7 \text{ kips}$$

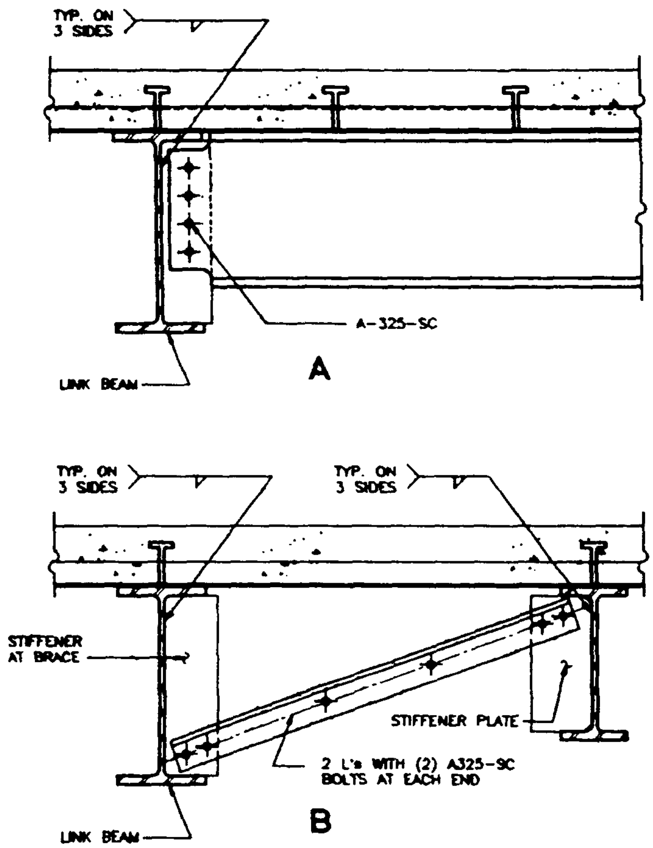


Figure 16.
Flange Bracing Options

The top flange is continuously braced by the metal deck. Figure 16 illustrates several options for bracing the lower flange. Similar details are typically used to brace the bottom flange of SMRFs per UBC 2211.7.8.

In Figure 16A the web stiffener is used to brace the lower flange. The stiffener transfers the brace load to the transverse purlin. The connection of the purlin to the web stiffener must be designed to transmit the horizontal shear of the brace load, the eccentric moment of the brace load between the lower beam flange and the purlin bolt group and the vertical shear from the gravity load on the purlin. UBC 1603.5 allows a one-third increase in the connection design capacity for the seismically induced brace load.

In Figure 16B a pair of angles are used to transfer the bracing load directly to the top flange of an adjacent parallel beam.

Beam bracing is required to prevent the length of unbraced portions of an EBF beam from exceeding $\frac{76b_f}{\sqrt{F_y}}$. A check for this condition was made, prior to the investigation of the influence of axial forces on the beam, to identify the weak axis unbraced length of the beam. In this example, beam bracing was not required outside the link for the W14 x 68 beams. However, beam bracing is required for the W12 x 50 beams. Their design is the same as for the link end bracing except that the bracing design force may be reduced. UBC 2211.10.18 requires lateral bracing resist 1.0% of the beam flange force at the brace point corresponding to 1.5 times the link beam strength. Conservative design of braces is recommended.

3.22 Brace to Beam Connection

UBC 2211.10.6 requires the connection to develop the compressive strength of the brace and transmit this force into the beam web. Extending the gusset plate or other connection components into the link could significantly alter the carefully selected section properties of the link. Therefore, no part of the connection is permitted to extend into the link length.

In this example, tube sections were used for the compression struts. Figure 17A illustrates a common link to brace detail. Tests have shown that this detail is susceptible to failure by severe buckling of the gusset plate (ref. 9, p. 508). Connection 17B is modified to minimize the distance from the end of the brace to the bottom of the beam. Some designers prefer to continue the gusset stiffener at the edge of the link along the diagonal edge of the gusset plate parallel to the brace. The gusset plate and the beam to gusset weld should be checked for stress increases when the axis of the brace force and the centroid of the weld do not coincide. The stress at the fillet of the beam web should be checked to see if a stiffener is required on the beam side of the brace to beam connection.

The center line axes of the brace and the beam typically intersect at the end of the link. This is not strictly necessary and may be difficult to achieve for various member size and intersection angle combinations. Moving this work point inside the link, as shown in Figure 17C, is acceptable (ref. 11, p. 332 C709.6).

Locating the work point outside the link as shown in Figure 17D tends to increase the bending in the link and may shift the location of the maximum combined bending and shear stress outside the link. However, the gusset of the beam to brace connection significantly increases the shear and bending capacity of the beam immediately adjacent to the link. Therefore, small movement of the work point outside the link may be acceptable; however, particular care should be used if this is done.

Any movement of the work point from the edge of the shear link should be accounted for in the analysis of the frame. An analytic model of the frame should be consistent with the work points. The link should be designed for the forces occurring within the relevant length of the analytic model.

The designer must take care to ensure that the location of maximum stress is inside the link and that the appropriate combinations of axial, flexural and shear stress are considered .

3.23 Brace to Column and Beam Connection

To remain consistent in the design, the connection of the brace to the column should develop the compressive strength of the brace. The detailing considerations for this connection are essentially the same as for a concentric brace. "Seismic Design Practice for Steel Buildings," (ref. 5, pp. 25, 26) illustrates some of the options available. A typical detail is shown in Figure 18A. The use of a large gusset plate welded in line with the beam and column webs will make this a moment connection. This type of beam to column connection should be analyzed with moment capacity. Stiffener plates have been used at the beam flange to column connection.

Figure 18B illustrates a bolted option for the brace to column connection. Horizontal stiffeners are used at the top, middle and bottom of the shear tab to prevent out-of plane twisting of the shear tab (ref. 8, p. 52). If the brace to beam connection work point shifts from the column centerline, as indicated, the moment produced by this offset must be included in the column design.

The beam to column connections shown in Figure 18 provide significant torsional restraint for the beam. UBC 2211.10.19 specifies the minimal torsional capacity for this connection.

3.24 Summary of Link and EBF Design

The design of the link portion of the beam is the most critical element of an EBF. As illustrated in the previous example, a link must provide for the following:

- Compact flanges and web
- Adequate shear capacity
- Adequate flexural and axial load capacity
- Limited rotation relative to the rest of the beam
- Limit drift of the EBF.

The design of an EBF is usually based on both stress and drift control including rotation angle. Both are equally significant. This is unlike the design of a moment frame where usually drift controls the design, or a concentrically braced frame where stress controls the design.

An EBF generally possesses excellent ductility, and it efficiently limits building drift. It may be a very cost effective bracing system.

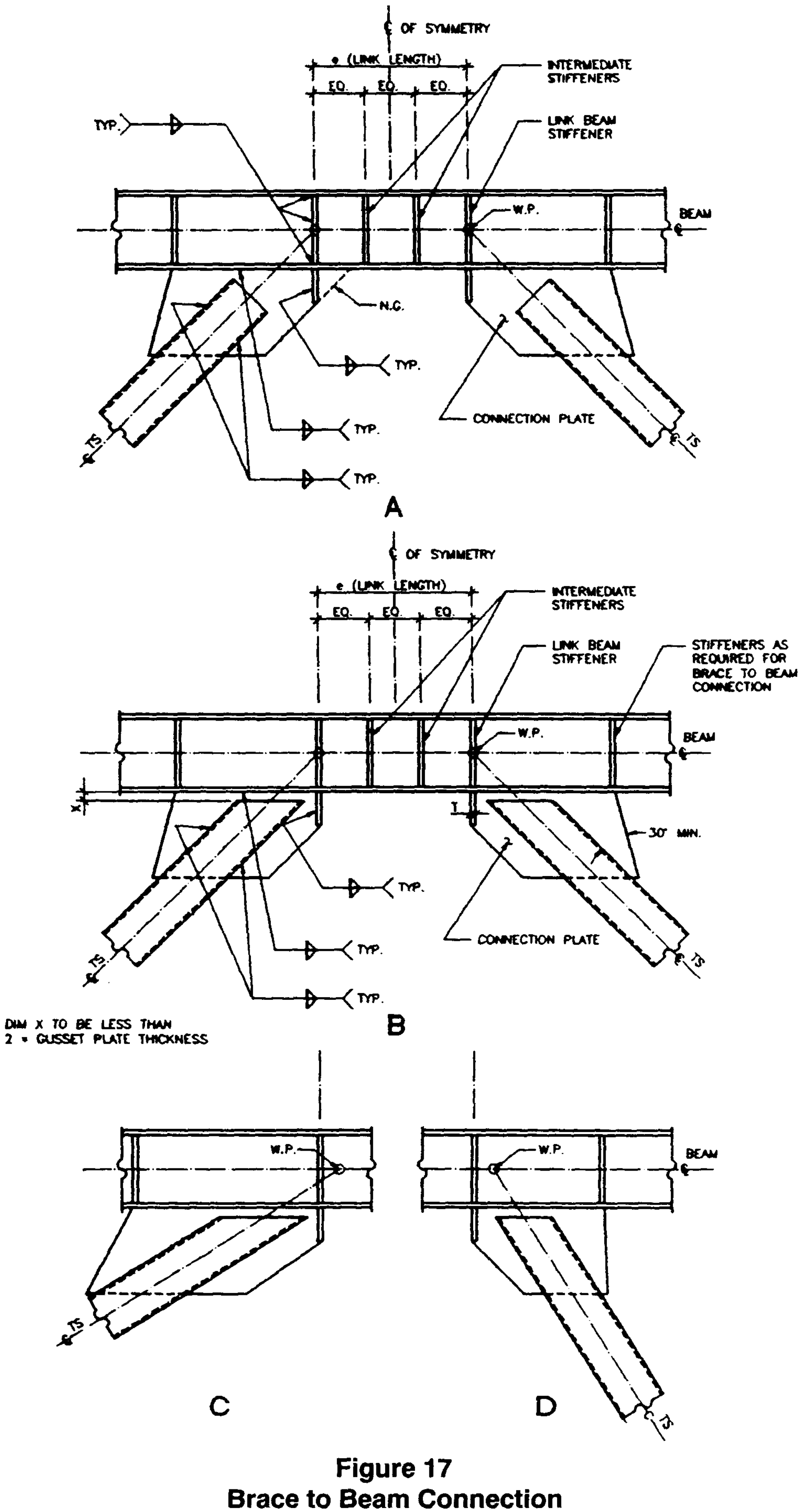
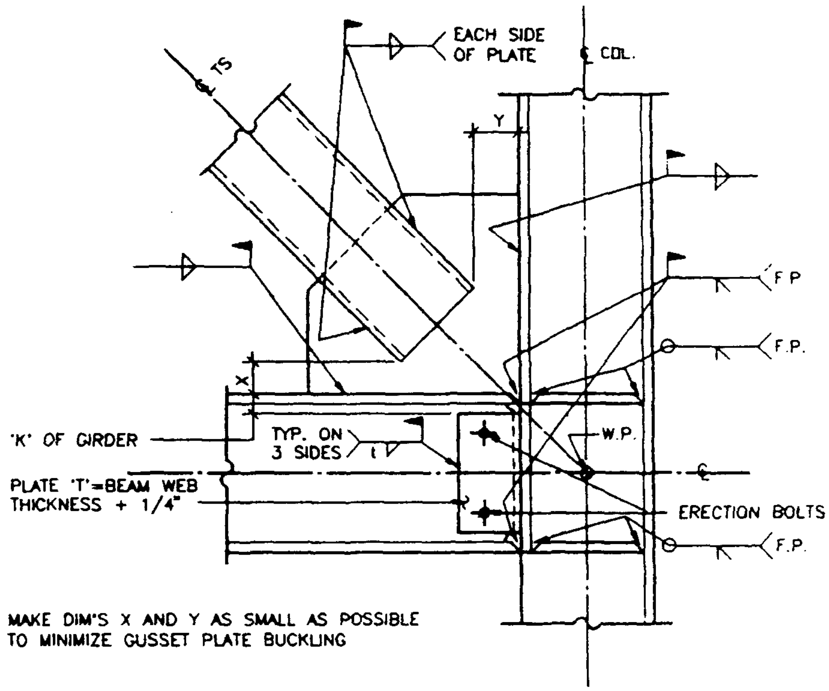
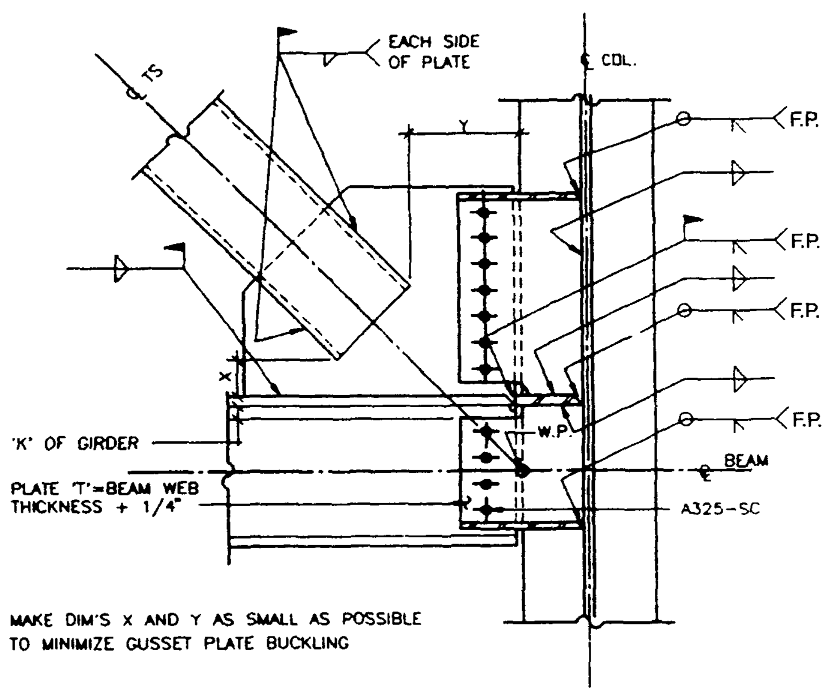


Figure 17
Brace to Beam Connection



A



B

Figure 18
Brace to Column and Beam Connections

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