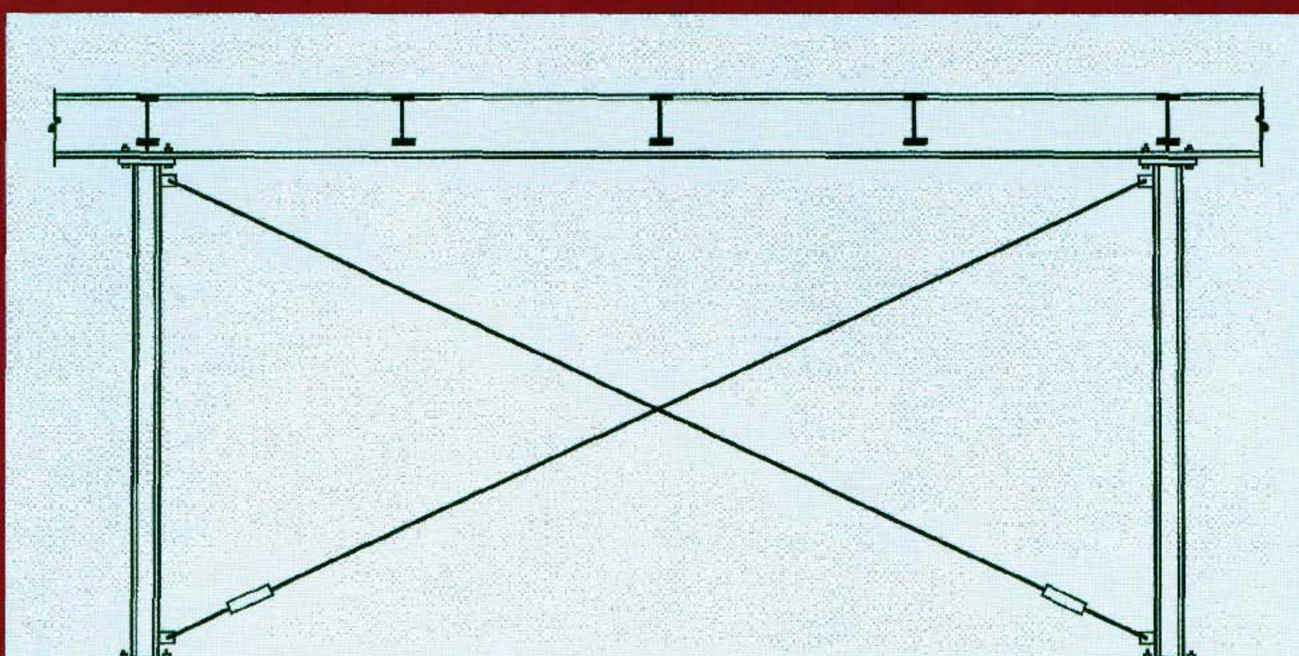
Steel Design Guide Series

10

Erection Bracing
of Low-Rise Structural Steel Buildings



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Erection Bracing

of Low-Rise Structured Steel Buildings

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TABLE OF CONTENTS

ERECTION BRACING OF LOW RISE STRUCTURAL STEEL BUILDINGS

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1. INTRODUCTION

This guide is written to provide useful information and design examples relative to the design of temporary lateral support systems and components for low-rise buildings. For the purpose of this presentation, low-rise buildings are taken to have the following characteristics:

(1) Function: general purpose structures for such uses as light manufacturing, crane buildings, warehousing, offices, and other commercial and institutional buildings.

(2) Proportions:

(a) height: 60 feet tall or less.

(b) stories: a maximum of two stories.

Temporary support systems are required whenever an element or assembly is not or has not reached a state of completion so that it is stable and/or of adequate strength to support its self-weight and imposed loads. The need for temporary supports is identified in Paragraph M4.2 of the AISC Specification for Structural Steel Buildings and in Section 7 of the AISC Code of Standard Practice for Steel Buildings and Bridges.

To a great extent the need for this guide on temporary supports was created by the nature and practice of design and construction of low-rise buildings. In many instances, for example, the lateral bracing systems for low-rise buildings contain elements which are not in the scope of the steel erector's work. For this reason the Code of Standard Practice makes a distinction between Self-Supporting and Non-Self-Supporting framework as will be discussed later. Other temporary supports such as shoring and cribbing for vertical loads are not included in the scope of this guide.

1.1 Types of Systems

Lateral bracing systems for low-rise buildings can be differentiated as follows:

Braced construction: In this type of system, truss-like bays are formed in vertical and horizontal planes by adding diagonals in vertical bays bounded by columns and struts or in horizontal bays bounded by beams and girders. In general, braced construction would be characterized as self-supporting, however, the frames may contain elements such as a roof deck diaphragm which would change the frame to a non-self-supporting type.

Rigid Frame Construction: This system uses moment resisting joints between horizontal and vertical framing members to resist lateral loads by frame action. In many buildings the rigid frames are discretely located within the construction to minimize the number of more costly moment resisting connections. The remainder of the frame would have simple connections and the frame would be designed to transfer the lateral load to the rigid frames. Rigid frame construction would also be characterized as self-supporting, however in the case of braced construction the framework may contain non-structural elements in the system which would make it a non-self-supporting frame.

Diaphragm Construction: This system uses horizontal and/or vertical diaphragms to resist lateral loads. As stated above horizontal diaphragms may be used with other bracing systems. Horizontal diaphragms are usually fluted steel deck or a concrete slab cast on steel deck. Vertical diaphragms are called shear walls and may be constructed of cast-in-place concrete, tilt-up concrete panels, precast concrete panels or masonry. Vertical diaphragms have also been built using steel plate or fluted wall panel. In most instances, the elements of diaphragm construction would be identified as non-self-supporting frames.

Cantilever Construction: Also called Flag Pole Construction, this system achieves lateral load resistance by means of moment resisting base connections to the foundations. This system would likely be characterized as self-supporting unless the base design required post erection grouting to achieve its design strength. Since grouting is usually outside the erector's scope, a design requiring grout would be non-self-supporting.

Each of the four bracing systems poses different issues for their erection and temporary support, but they share one thing in common. All as presented in the project Construction Documents are designed as complete systems and thus all, with the possible exception of Cantilever Construction, will likely require some sort of temporary support during erection. Non-self-supporting structures will require temporary support of the erection by definition.

1.2 Current State of the Art

In high-rise construction and bridge construction the need for predetermined erection procedures and temporary support systems has long been established in the industry. Low-rise construction does not command a comparable respect or attention because of the low heights and relatively simple framing involved. Also the structures are relatively lightly loaded and the framing members are relatively light. This has lead to a number of common fallacies which are supported by anecdotal evidence.

1.3 Common Fallacies

1. Low-Rise frames do not need bracing. In fact, steel frames need bracing. This fallacy is probably a carryover from the era when steel frames were primarily used in heavy framing which was connected in substantial ways such as riveted connections.

2. Once the deck is in place the structure is stable. In fact, the steel deck diaphragm is only one component of a complete system. This fallacy obviously is the result of a misunderstanding of the function of horizontal diaphragms versus vertical bracing and may have resulted in the usefulness of diaphragms being oversold.

3. Anchor rods and footings are adequate for erection loads without evaluation. In fact, there are many cases in which the loads on anchor rods and footings may be greater during erection than the loads imposed by the completed structure.

4. Bracing can be removed at any time. In fact, the temporary supports are an integral part of the framework until it is completed and self-supporting. This condition may not even occur until some time after the erection work is complete as in the case of non-self-supporting structures.

5. The beams and tie joists are adequate as struts without evaluation. In fact, during erection strut forces are applied to many members which are laterally braced flexural members in the completed construction. Their axially loaded, unbraced condition must be evaluated independently.

6. Plumbing up cables are adequate as bracing cables. In fact, such cables may be used as part of temporary lateral supports. However, as this guide demonstrates additional temporary support cables will likely be needed in most situations. Plumbing a structure is as much an art as a science. It involves continual adjustment commonly done using diagonal cables. The size and number of cables for each purpose are determined by different means. For example, the lateral support cables would likely have a symmetrical pattern whereas the plumbing up cables may all go in one direction to draw the frame back to plumb.

7. Welding joist bottom chord extensions produces full bracing. In fact, the joist bottom chords may be a component of a bracing system and thus welding them would be appropriate. However, other components may be lacking and thus temporary supports would be needed to complete the system. If the joists have not been designed in anticipation of continuity, then the bottom chords must not be welded.

8. Column bases may be grouted at any convenient time in the construction process. In fact, until the column bases are grouted, the weight of the framework and any loads upon it must be borne by the anchor rods and leveling nuts or shims. These elements have a finite strength. The timing of grouting of bases must be coordinated between the erector and the general contractor.

1.4 Use of This Guide

This guide can be used to determine the requirements for temporary supports to resist lateral forces, i.e. stability, wind and seismic. The guide is divided into two parts. Part 1 presents a method by which the temporary supports may be determined by calculation of loads and calculation of resistance. Part 2 presents a series of prescriptive requirements for the structure and the temporary supports, which if met, eliminate the need to prepare calculations. The prescriptive requirements of Part 2 are based on calculations prepared using the principles presented in Part 1.

PART1

DETERMINATION OF BRACING REQUIREMENTS BY CALCULATION METHOD

2. INTRODUCTION TO PART 1

Part 1 consists of three sections. The first deals with design loads which would be applicable to the conditions in which the steel framework exists during the construction period and specifically during the period from the initiation of the steel erection to the removal of the temporary supports. Sections 4 and 5 deal with the determination of resistances, both of permanent structure as it is being erected and of any additional temporary supports which may be needed to complete the temporary support system. An appendix is also presented which provides tabulated resistances to various components of the permanent structure. This appendix follows the reference section at the end of the guide.

3. CONSTRUCTION PHASE LOADS FOR TEMPORARY SUPPORTS

The design loads for temporary supports can be grouped as follows:

Gravity loads
- Dead loads on the structure itself
- Superimposed dead loads
- Live loads and other loads from construction operations

Environmental loads
Wind
Seismic

Stability loads

Erection operation
Loads from erection apparatus
Impact loads caused by erection equipment and pieces being raised within the structure

3.1 Gravity Loads

Gravity loads for the design of temporary supports consist of the self-weight of the structure itself, the self-weight of any materials supported by the structure and the loads from workers and their equipment. Self-weights of materials are characterized as dead loads. Superimposed loads from workers and tools would be characterized as live loads. Gravity loads can be distributed or concentrated. Distributed loads can be linear, such as the weight of steel framing members, non-uniform such as concrete slabs of varying thicknesses or uniform such as a concrete slab of constant thickness.

Dead loads can be determined using the unit density and unit weights provided in the AISC Manual of Steel Construction, (LRFD Part 7, ASD Part 6) and ASCE 7-93, Tables Cl and C2. Dead loads can also be obtained from manufacturers and suppliers.

Live loads due to workers and their equipment should be considered in the strength evaluation of partially completed work such as connections or beams which are unbraced. The live load used should reflect the actual intensity of activity and weight of equipment. In general, live loads on the order of 20 psf to 50 psf will cover most conditions.

3.2 Environmental Loads

The two principal environmental loads affecting the design of temporary supports are wind and seismic loads. Other environmental loads such as accumulated snow or rain water may influence the evaluation of partially completed construction but these considerations are beyond the scope of this guide.

3.2.1 Wind Loads

Wind loads on a structure are the result of the passage of air flow around a fixed construction. The load is treated as a static surface pressure on the projected area of the structure or structural element under consideration. Wind pressure is a function of wind velocity and the aerodynamic shape of the structure element. Various codes and standards treat the determination of design and wind pressures slightly differently, however the basic concept is common to all methods. What follows is a discussion of the procedure provided in ASCE 7-93 (1) which will illustrate the basic concept.

In ASCE 7-93 the basic design pressure equation for the main force resisting system for a building is

$$p = qG_hC_p - qh(GC_{pi}) \qquad \text{Eq.3-1}$$

where

$$q - 0.00256K(IV)^2 \qquad \text{Eq. 3-2}$$

K = velocity pressure coefficient varying with height and exposure

Exposure classes vary from A (city center) to D (coastal areas) and account for the terrain around the proposed structure.

I = an importance factor which varies with the use of the building, for design of temporary supports I may be taken as 1.0 without regard to the end use of the structure

V = the basic wind speed for the area taken from weather data, usually a 50 year recurrence interval map

G_h = a factor accounting for gust response varying with horizontal exposure

C_p = a factor accounting for the shape of the structure

q_h = q taken at height, h

GCpi = a factor accounting for internal pressure

This method or one like it would have been used to determine the wind forces for the design of the lateral force resisting system for a structure for which temporary lateral supports are to be designed.

To address the AISC Code of Standard Practice requirement that "comparable" wind load be used, the same basic wind speed and exposure classification used in the building design should be used in the design of the temporary supports.

The design of temporary supports for lateral wind load must address the fact that the erected structure is an open framework and as such presents different surfaces to the wind.

In ASCE 7-93 the appropriate equation for open structures is:

$$p = q_zG_hC_f \qquad \text{Eq. 3-3}$$

where

q_z = q evaluated at height z

G_h = gust response factor G evaluated at height, h, the height of the structure

C_f = a force coefficient accounting for the height and aerodynamic geometry of the structure or element

The current draft of the ASCE Standard "Design Loads on Structures During Construction" provides a reduction factor to be applied to the basic wind speed. This factor varies between 1.0 for an exposure period more than 25 years and 0.75 for an exposure period of less than six weeks. The factor for an exposure period from 6 weeks to one year is 0.8.

To determine a wind design force, the design pressure, p, is multiplied by an appropriate projected area. In the case of open structures, the projected area is an accumulated area from multiple parallel elements. The accumulated area should account for shielding of leeward elements by windward elements. Various standards have provided methods to simplify what is a rather complex aerodynamic problem. The elements of the multiple frame lines can be solid web or open web members. Thus, the determination of wind forces requires an evaluation to determine the correct drag coefficient and the correct degree of shielding on multiple parallel members. It also requires the correct evaluation of the effects of wind on open web members.

This topic has been treated in the following documents:

1. Part A4.3.3 of the "Low Rise Building Systems Manual" (12) published by the Metal Building Manufacturers Association.

2. "Wind forces on Structures" (18), Paper No. 3269, ASCE Transactions, published by the American Society of Civil Engineers.

3. "Standards for Load Assumptions, Acceptance and Inspection of Structures" (16), No. 160, published by the Swiss Association of Engineers and Architects.

4. "Design Loads for Buildings" (5), German Industrial Standard (DIN) 1055, published by the German Institute for Standards.

Perhaps the most direct method is that given in the current draft of the ASCE Standard for Design Loads on Structures During Construction which states:

> "6.1.2. Frameworks without Cladding
> Structures shall resist the effect of wind acting upon successive unenclosed components.
>
> Staging, shoring, and falsework with regular rectangular plan dimensions may be treated as trussed towers in accordance with ASCE 7. Unless detailed analyses are performed to show that lower loads may be used, no allowance shall be given for shielding of successive rows or towers.
>
> For unenclosed frames and structural elements, wind loads shall be calculated for each element. Unless detailed analyses are performed, load reductions due to shielding of elements in such structures with repetitive patterns of elements shall be as follows:
>
> 1. The loads on the first three rows of elements along the direction parallel to the wind shall not be reduced for shielding.
>
> 2. The loads on the fourth and subsequent rows shall be permitted to be reduced by 15 percent.
>
> Wind load allowances shall be calculated for all exposed interior partitions, walls, temporary enclosures, signs, construction materials, and equipment on or supported by the structure. These loads shall be added to the loads on structural elements.
>
> Calculations shall be performed for each primary axis of the structure. For each calculation, 50% of the wind load calculated for the perpendicular direction shall be assumed to act simultaneously."

In this procedure one would use the projected area of solid web members and an equivalent projected area for open web members. This effective area is a function of the drag coefficient for the open web member which is a function of the solidity ratio. For the types of open web members used in low-rise construction an effective area (solidity ratio, (p) equal to 30 percent of the projected solid area can be used.

Shielding of multiple parallel elements can be determined using the following equation taken from DIN 1055. See Figures 3.1 and 3.2.

$$A = (1+\eta+(n-2)\eta^2)A_1 \qquad \text{Eq. 3-4}$$

where

A = total factored area

η = a stacking factor taken from Figure 3.2.

n = the total number of parallel elements

A_1 = the projected area of one element

The stacking factor, η, is a function of the element spacing to the element depth and a solidity ratio, φ.

3.2.2 *Seismic Loads*

As indicated in the AISC Code of Standard Practice, seismic forces are a load consideration in the design of temporary supports. In general, seismic forces are addressed in building design by the use of an equivalent pseudo-static design force. This force is a function of:

1. an assessment of the site specific seismic likelihood and intensity,

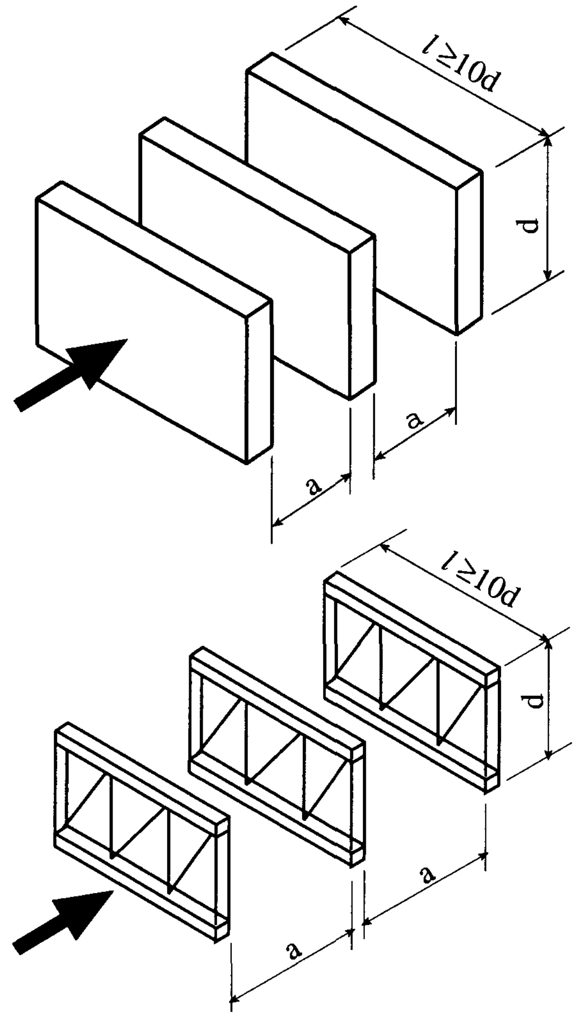

Fig. 3.1 Parameters for Use with Fig. 3.2

2. the use of the structure,

3. the geometry and framing system type of the structure,

4. the geological nature of the building site, and

5. the mass, i.e. self-weight of the structure.

Although codes and standards have differing approaches to seismic design, they are conceptually similar. The general approach can be seen in the description of the approach used in ASCE 7-93 which follows.

The general equation for seismic base shear, V, is:

$$V = C_s W \qquad \text{Eq.3-5}$$

where

C_s = the seismic design coefficient

W = the total dead load and applicable portions of other loads

For the structures within the scope of this guide it is unlikely that W would include any loads other than dead load.

The seismic design coefficient, C_s, is to be determined using the following equation:

$$C_s = \frac{1.2A_vS}{RT^{2/3}} \qquad \text{Eq. 3-6}$$

where

A_v = a coefficient representing the peak velocity related acceleration taken from a contour map supplied

S = a coefficient for site soil profile characteristics ranging from 1.0 to 2.0

R = a response modification factor, ranging from 1.5 to 8.0 depending on the structural system and the seismic resisting system used

T = the fundamental period of the structure which can be determined using equations provided

ASCE 7-93 states that the seismic design coefficient, C_s, need not exceed the value given by the following equation:

$$C_s = \frac{2.5A_a}{R} \qquad \text{Eq. 3–7}$$

where

A_a = a coefficient representing the effective peak acceleration taken from a contour map supplied

R = the response modification factor described above

For the structures within the scope of this guide the response modification factor, R, would be 5.0. This value for R_w is taken from ASCE 7, Table 9.3-2 and is the value given for "Concentrically-braced frames". Likewise for the majority of regular structures there is not significant penalty in using the simpler equation given above to determine C_s. The range of values in the contour map provided in ASCE 7-93 are 0.05 through 0.40. Thus, the range of values for C_s is 0.025 to 0.20. In general wind will govern the design of temporary supports in areas of low seismic activity such as the mid-west. Seismic forces will likely govern the design on the west coast. The value of A_a would be the same value used in the design of the completed structure. Although this discussion of the determination of C_s would apply to most structures in the scope of this guide, it is incumbent on the designer of the temporary support system to be aware of the requirements for seismic design to confirm that the general comments of this section apply to the specific structure at hand.