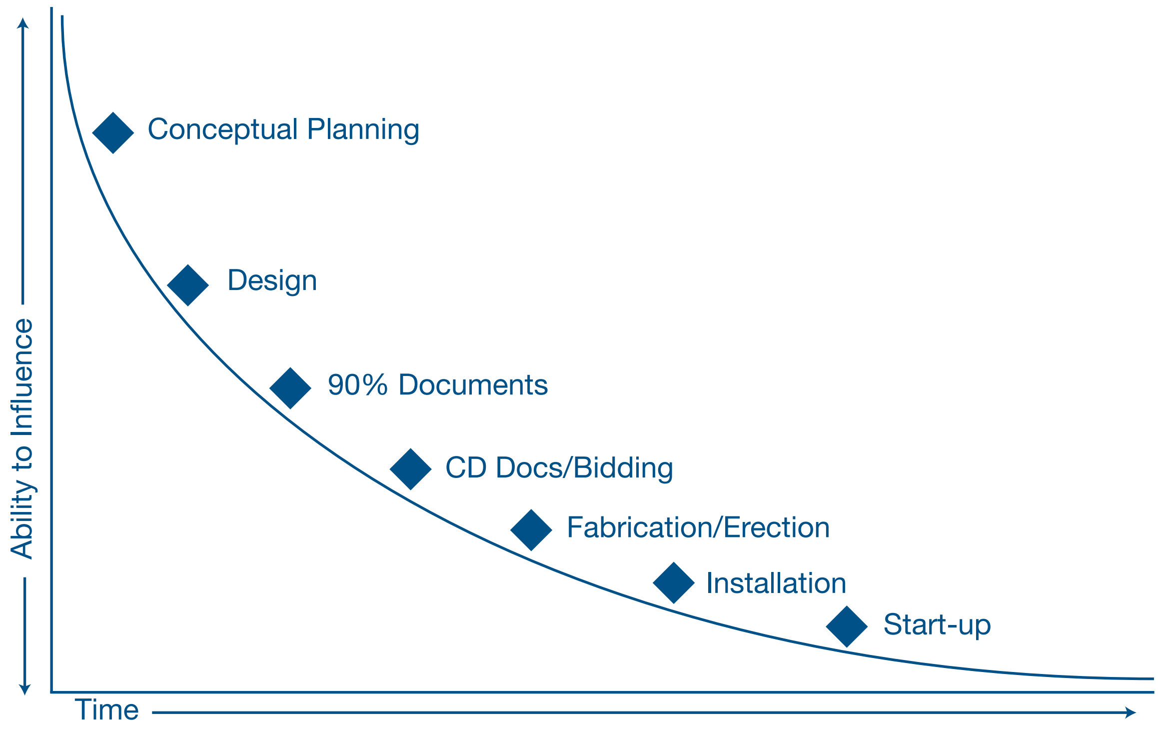
23

Steel Design Guide

Constructability of Structural Steel Buildings



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Constructability of Structural Steel Buildings

DAVID I. RUBY, P.E., S.E.
Ruby + Associates
Farmington Hills, Michigan

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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Printed in the United States of America

PUBLISHER'S NOTE

This document differs in use and application from many previous AISC publications. It is based upon evolving thought on new project delivery systems in the industry and addresses concepts that are appearing in the professional literature on an increasing basis. The author's ideas involve all construction trades and design disciplines, not just structural engineers and structural steel fabricators, and this document can serve as a primer for structural engineers and others in the structural steel industry who seek new approaches to construction and new ways of doing business.

While the terms are not used explicitly, the author's recommendations very much parallel the concepts of integrated project delivery, lean construction, and alliance contracting. In many respects the concepts in this Design Guide are ahead of many industry theorists—with one important difference. The author is not just theorizing about integrating "constructability" into his structural engineering practice. Rather, he has actually done it and is sharing his knowledge with colleagues and the industry, which he has served well for many years.

This Design Guide does not constitute a code or standard; nor is it intended to be incorporated by reference into a contract document. However, it has tremendous potential utility in guiding an evolving practice and standard of care in an era when new contract documents and contract relationships are being developed to address some of the concerns raised in this text.

Several distinguishing characteristics of this work should be kept in mind as its principles are applied to current and future real-world construction projects:

1. Some of the practice suggestions addressed are clearly within the recognized, traditional province of the Structural Engineer of Record.
2. Some of the practice suggestions addressed are applied by some structural engineers, but not by all practitioners—or even a majority of practitioners—and therefore have not risen to the level of either "standard practice" or a recognized standard of care.
3. Some of the suggestions addressed are either "means and methods" of construction or matters that, under current project delivery systems, can only be addressed by the owner or the prime design professional (usually the project architect).
4. Because this text is not constrained by traditional thought and traditional approaches, it does not differentiate among the categories of traditional practice or the traditional professional responsibility that is applied to those categories of practice by different members of the project team. Therefore, this work should not be used in an attempt to define the professional responsibility of any individual member of a project team.
5. Finally, this text covers a great deal of technical information. It is an extremely valuable tool, but cannot be applied in a vacuum, or by someone who does not have the prerequisite level of technical training and experience. It has to be applied simultaneously by a host of qualified professionals, working together, using the references noted, and a good many additional references that may not necessarily be noted.

TABLE OF CONTENTS

Preface

Productivity and innovation within the construction industry are lagging far behind the gains experienced in the manufacturing industry. Businesses and trade organizations engaged in construction contribute little to research and development to improve the process.

The design, fabrication and installation process is far too fragmented. There is little opportunity for mass production or repetitive work, in part because we are associated with the custom fabrication industry. Moreover, traditional design and construction methods are often self-protective and based in adversarial relationships.

This lack of innovation and integration reaches across the entire design community and construction industry—increasing costs, affecting our image, and reducing prosperity. If our industry fails to prosper, it will no longer invest in itself. Stagnation will occur and innovation will be further stifled, resulting in a cycle of disincentive and decline.

Productivity matters to every engineer, contractor and owner because it provides the essential ingredient that makes nations rich. When companies produce more for each hour their employees work, they can pay higher wages and reap bigger profits. An annual productivity growth of 2% would more than double inflation-adjusted wages over 40 years, all else being equal. Add another percentage point in productivity growth, and wages would more than triple (Whitehouse and Aeppel, 2006).

Over the last decade, innovation through information technology has been the driver of productivity for the financial, health care and manufacturing industries. The construction industry, on the other hand, has not taken full advantage of this technology.

While manufacturing has embraced robotic and computer technology, what presently occurs at most construction sites has changed little over the years. The fabrication industry uses computer-aided estimating and advanced bill of materials production, automated beam lines, computer detailing, and digitized plasma cutting. Cell phones and laptop computers have improved field communication. Advances in software have made it possible to create sophisticated scheduling and document tracking programs, and better construction equipment has resulted in small productivity increases. Nevertheless, these technological innovations have not changed the fundamental way in which projects are planned, designed and built.

The construction industry has primarily considered information technology as a replacement for the pencil, drafting board and shop drawing, without entertaining such questions as to how better to use these tools to improve the process and integrate the process of steel design, fabrication and installation.

Constructability answers these questions. Constructability as a design concept can be the initial step in the integration of the process and will enable the design professional to develop creative solutions and bring enhanced value to the client. This design guide outlines the fundamentals of constructability and offers suggestions on implementation of the concept.

The author thanks the following individuals for their contributions to this design guide: Lawrence A. Kloiber, Lawrence F. Kruth, Robert E. Shaw, Jr., William H. Treharne, Thomas D. Wosser, Brian M. Volpe, Carlo Lini, and Allison Shenberger. Also, the author thanks the following reviewers for their comments and suggestions:

William A. Andrews
Charles J. Carter
Don Engler
Lawrence F. Kruth
Keith Landwehr
Brett R. Manning
R. Shankar Nair
Davis G. Parsons II
Victor Schneur
James A. Stori
Emile W.J. Troup
Kenneth B. Wiesner
Ronald G. Yeager

Chapter 1
Introduction

The Construction Industry Institute (CII, 1993) defines constructability as the optimum use of construction knowledge and experience in planning, design, procurement and field operations to achieve overall project objectives. Those who advocate this concept believe that constructability can bring real benefits to all involved—clients, consultants and contractors. Benefits include enhanced cooperation, reduced risk, improved schedule, budget control, and elimination of litigation.

Constructability includes visualizing the construction of the project prior to beginning the actual design, and maintaining that vision throughout the design process. The focus is on maximizing simplicity, economy, and speed of construction, while considering such project-specific factors as site conditions, code restrictions and owner requirements. Constructability is a design philosophy that begins in the conceptual design stage, continues through design, and links project planning with design and construction.

Constructability can be a challenge. The traditional approach separates the individual functions involved in planning, design, procurement and construction into specific tasks—each performed by specific parties. Planning is often performed by the architect, with systems design prepared by the engineers. Procurement is managed by the construction manager, and construction is performed by the general contractor and appropriate trades.

The steel industry also follows this traditional process. The structural design is typically separated from the detailing, fabrication and erection, which are also normally separate and distinct functions. The design professional tends to place emphasis on the design, budget, schedule and liability, while the detailer concentrates on shop and erection drawing preparation, and the fabricator and erector separately concentrate on their respective roles in meeting the project schedule and budget. These diverse interests, pitting design versus fabrication versus erection, are not beneficial to the owner.

Constructability seeks to integrate this process and reap the benefits of collaboration. It is an approach that infuses construction knowledge and experience into the design process, creating a project that achieves the overall project objectives while reducing costs, improving the schedule, and eliminating litigation. This will create satisfied designers, builders and owners!

1.1 HOW DOES CONSTRUCTABILITY HAPPEN?

While many design professionals have significant knowledge about what makes a project constructible, benefit can almost always be derived from the early involvement of a steel contractor or a constructability consultant. Input in the planning and conceptual stages of a project provides for a more informed decision-making process based upon accurate and up-to-date cost estimates and value engineered suggestions. In addition, design document reviews, subcontractor qualifications, site constraints, weather impact, and schedule concerns can be evaluated sooner, thereby making the number of alternatives that can be considered larger.

Four common characteristics essential to achieving constructability are (CII, 1986a):

1. The owner and managers of the design and construction teams are committed to the concept of constructability and openly share knowledge and experience for the benefit of the project.

2. Constructability considerations are used in determining project cost and schedule objectives.

3. The early involvement of experienced construction personnel is used to foster full understanding of the planning, design and construction processes to be used for the project.

4. Communication works both ways between the design team and construction team with all participants thinking about constructability, requesting input freely, and evaluating that input objectively.

It is usually a proactive design professional who educates the owner about the benefits of early involvement of industry professionals in the design process. Note that the owner who will benefit from constructability may need to go beyond conventional approaches to project execution by expanding front-end planning and investing additional time, effort and money to discover opportunities and anticipate potential problems. This up-front money will almost always pay dividends later with reduced total project cost and/or schedule. Construction Industry Institute research (CII, 1986b) indicates that cost reductions of at least 6%—and as high as 23%—are possible with benefit/cost ratios as high as 10 to 1.

In addition, constructability considerations usually will offer significant reductions in the project schedule.

To achieve the benefits of constructability, the early involvement of an industry professional is key. This industry professional may be one or more of many different types, including:

- Specialty Structural Engineer—a structural engineer who specializes in structural steel design and construction.
- Contractor Engineer—a structural engineer, employed by a fabricator, erector, or other steel-savvy contractor, who has extensive experience in steel design and construction.
- Connection Designer—a structural engineer who specializes in steel connection design with extensive experience in structural steel fabrication and construction.
- Independent Consultant—a structural engineer who has extensive experience in the structural steel industry.

1.2 IS CONSTRUCTABILITY THE SAME AS VALUE ENGINEERING?

No, constructability and value engineering differ; however, many constructability concepts are used in the typical value engineering review, including:

1. Consideration of site constraints, local labor skills, and material availability.
2. Maximizing framing efficiency, with consideration of the fabrication and erection processes.
3. Review of design documents at the various stages of development, including post-design reviews by field personnel.
4. Consideration of modular construction or shop assemblies to optimize the completed project.
5. Simplification of framing and connection details.

Note the importance of early involvement by the industry professional in these activities.

While value engineering may provide some savings, it is by nature a process that only fine-tunes the individual parts. As such, it cannot achieve a finely tuned project. In contrast, constructability integrates the process by engaging all the players at the earliest possible stage, jointly developing a qualified and cooperative design and construction team in a collaborative process, thus taking the optimum advantage of the available construction knowledge and experience.

As illustrated in Figures 1-1 and 1-2, maximum ability to influence occurs when constructability is considered during the earliest stages of the project, and these changes are most cost-effective early on. Constructability maximizes benefits to the owner by affecting the total project, starting in the early planning and design phases when industry knowledge and experience is infused into the design process. In contrast, value engineering is more commonly performed after substantial design decisions have been made. Not only is this too late to make changes that would maximize the benefit to the owner, it fosters a perception that the suggestions are a criticism of the designer, self-serving for the fabricator or erector, and too little too late. Simply stated, value engineering occurs when there is limited opportunity to truly impact the project cost or schedule.

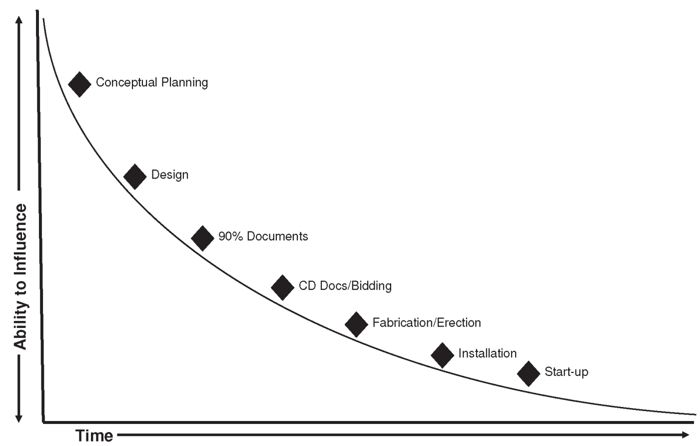

Fig. 1-1. Ability to influence over project life.

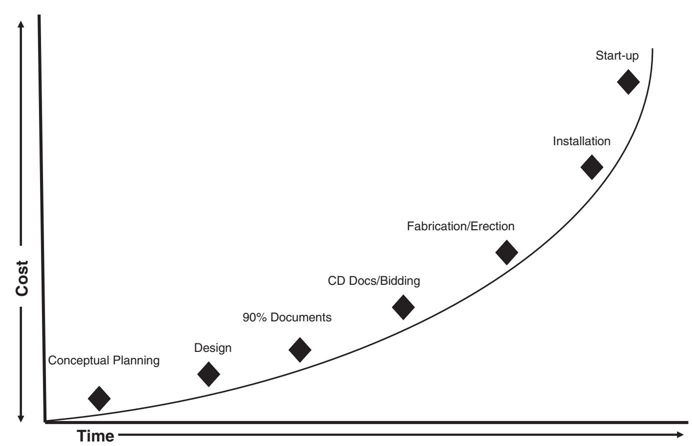

Fig. 1-2. Cost of change over time.

1.3 FUNDAMENTALS OF CONSTRUCTABILITY

Constructability is not a magic pill and it cannot be successful without the commitment of the entire design team. Constructability is a process, not an event. Constructability as a design approach includes all of the elements that drove us to become engineers, that continue to bring excitement and generate enthusiasm in our daily lives, and that engage us in developing solutions for those near impossible situations. This process is a design philosophy that requires:

- Collaboration and coordination—designer-constructor communication that includes all designers and construction trades.
- Visualization to jointly develop and maintain a vision of the project throughout the process.
- Innovation and imagination—a clear, concise project vision including the concept, attributes and constraints, and a clean-slate concept development with designers actively seeking and incorporating construction input.
- Integration from concept development to construction completion to occupancy.

Constructability assists in initial project scheduling by developing plans that work for both design and construction, while recognizing the opportunities and accounting for the realities of the actual project. The sequence and completion schedule for development of the concept and design can be structured to permit an efficient work plan with coordinated delivery and installation sequences. The project plan can be created with construction durations that are feasible and include allowances for potential weather conditions. Local conditions, which could create opportunities for innovative solutions or generate major production problems, can be recognized and addressed.

The site layout is often a key determining factor when making constructability decisions. Commercial buildings often maximize the use of space within the governing code provisions. A restricted site creates challenges in construction, such as adequate areas for lay-down and subassembly, shakeout or project sequencing, personnel access, and material delivery. The site may also limit installation methods and/or equipment and require more coordination among contractors and subcontractors. In contrast, process and plant operations generally dictate the site layout for industrial projects. These layouts are the product of standard industry clearances and work station layouts, which are not always compatible with the structural requirements.

The selection of the basic structural system may require several iterations from initial concept to final design. Such iterations are a vital step in developing potential savings and reduced risk for the owner. The early involvement of an industry professional can greatly assist with this process. Opportunities for cost or schedule savings can be identified, such as when high-strength steel should be considered, what materials are readily available and on what schedule, what connections might best serve the design and construction of the project, and how shop fabrication can be maximized. During the iterative design development stage, the determination of the structural concept should be based on proven structural systems, specific project constraints, known industry standards, and consideration of the available fabrication and installation processes. In addition, methods to accommodate such considerations as distortion, temperature effects, elastic shortening, weld shrinkage and erection aids should also be considered as early as possible in the project planning.

All projects will benefit from constructability input, which provides the right balance between production requirements and building constraints. Early involvement will foster innovation, improve the basic structural design, and also reduce or eliminate the potential for problems.

1.4 BUILDING INFORMATION MODELING

Building information modeling, commonly called BIM, is the compilation of construction and design information graphically represented and housed in a database. The concept embraces many of the attributes of constructability—cooperation, collaboration, integration and visualization. BIM allows conflicts to be detected during the design process rather than in the field by the trades.

Like constructability, BIM benefits those in the design and construction fields, as well as the owner. Sophisticated facility owners have benefited in the past from constructability reviews and are now realizing the benefits of BIM. They recognized that higher initial design costs make for drastically reduced schedules, lower construction costs, fewer change orders, and lower facility maintenance costs.

A BIM model can enhance the constructability input by providing three-dimensional (3-D) visualization during the various stages of the design process, as well as assisting with initial coordination and the development of drawings, sections and details. BIM can also assist with project estimating and scheduling, visualization, and interference checking. Again, these are all aspects of constructability.

However, BIM is not a magic pill. It is a tool with a real value that comes only through integration with the concept of constructability during the design process. Without construction knowledge and experience, the result may be less than optimized. BIM is not just an amalgamation of design technologies that represent every building component in a virtual environment; nor is it a 3-D rendering of a building.

BIM should be viewed as a project delivery method, enhanced by constructability, with new risks, rewards and relationships. New business models will be developed to integrate the new technologies into professional practices, and these business models will require cooperation and collaboration of all parties involved.

When working with a virtual building, team members are assembled and reassembled numerous times to collaborate, coordinate and resolve conflicts and clashes. Because all aspects of a project are driven from a single database or related databases, issues of drawing coordination and conflict errors are greatly diminished. Integration of information from multiple disciplines also supports the concept of constructability.

Deep collaboration promises greatly increased efficiency and quality. BIM is most effective when the key participants, designers and construction professionals are jointly involved in developing and augmenting the central model. Although traditional roles remain, the transitions between participants are less abrupt and less easily defined.

1.5 IMPLEMENTATION OF CONSTRUCTABILITY

According to P. Douglas Folk, Esq. (Folk, 2005), over half of *Engineering News Record*'s Top 500 Design Firms have formalized a corporate philosophy promoting constructability within their firms (25% throughout the entire design process and 51% as early as the conceptual planning stage). By adopting a constructability design philosophy, structural engineers can seize the opportunity to position the profession for the future and improve performance on projects today.

Constructability is a design philosophy that positions the structural engineering profession to be a continuing asset to the construction community through the integration of planning, design and construction throughout the concept development, design/BIM and construction processes.

Chapter 2
Early Involvement

Constructability discussions must begin early in the design phase to maximize benefit. Such discussions should include the design team, the owner's designated representative for construction, and as many project participants with structural steel construction knowledge and experience as possible, including the steel fabricator, the steel erector, a constructability consultant, or others.

The ultimate success of a project is directly related to the quality and makeup of the design team and their dedication to the successful implementation of the constructability concept. The design team should consist of competent professionals from both the design team and construction industry, experienced in the design and construction of related or similar projects. Design professionals with construction experience and knowledge are a significant asset to the process. In lieu of the availability of such experience, the design team is strongly encouraged to seek out like-qualified construction-related personnel and invite them to join the team. Such experience cannot be measured in dollars and cents. It is also helpful to include other design professionals and trade contractors whose work will affect or be affected by the structural work.

2.1 INITIAL PLANNING DECISIONS

The decisions made during the conceptual and planning stages establish a limited-access "highway" for all future decisions. Often these decisions are made based on historical data, an owner's request or directive, or the whim of an influential project decision maker. Even worse, decisions are often made without input from professionals experienced and knowledgeable in structural steel construction and its related costs. Constructability seeks to change this.

Early discussions should include a professional who is experienced and knowledgeable in steel construction. Ideally, this will be a member (or members) of the structural steel construction team, or an industry professional (see Chapter 1). Discussion topics should include:

- A review and understanding of the owner's wants, needs, constraints and goals.
- A review of the architect's proposed concepts and project program.
- A review of the design schedule and milestones for coordination and completion of the construction documents (topics that are discussed further below).
- Evaluation of site access, surface conditions, site constraints and similar factors.
- A review of building code requirements, including the environmental issues applicable to the project.
- A review of soil conditions, seismic site class, and the recommendations of the geotechnical engineer.
- Discussion of seismic design requirements and their impact on the costs and schedule of design, detailing, fabrication, erection and inspection.
- Discussion of performance-based design opportunities—a structural design that is commensurate with the risk (or avoidance of risk) that the owner is willing to accept.
- Consideration of new and developing technologies in materials, design and construction that could have favorable impact on project budget or schedule.
- A review of special architectural requirements, if any, such as coating requirements, architecturally exposed steel (AESS) requirements, special tolerances, etc.
- A review of communication procedures, intended to foster open and free communication, especially among key interfacing parties that may not have formal contractual obligations, such as the structural engineer of record (SER) and steel detailer.
- Evaluation of foreseeable factors that are within the control of the parties involved on the project, such as jobsite safety, quality, budget, or construction schedule.
- Evaluation of foreseeable factors that are not within the control of the parties involved on the project, such as sudden material price escalation, material availability, bad weather, unanticipated soil conditions, or a poorly performing subcontractor.

These initial decisions, influenced by constructability, will establish the program and acceptance criteria for the subsequent design decisions on the project.

2.2 COORDINATION AND COMPLETENESS OF CONSTRUCTION DOCUMENTS

Coordinated construction documents are those that separate an efficient relationship of different elements of the whole project into the various common disciplines such that the various elements of the project coalesce and perform as a system in the completed project. Complete construction documents are those that contain a sufficient level of information to allow a competent contractor to accurately price the project and, upon award, build the project in a manner consistent with the scope of the documents at the time of bidding. The coordination and completeness of structural design documents is discussed at considerable length in CASE Document 962D, *A Guideline Addressing Coordination and Completeness of Structural Construction Documents* (CASE, 2003), hereafter referred to as CASE 962D. Included in that document are critical discussion topics for the owner, architect, SER and construction team during the planning stage. As described in CASE 962D, the impact of coordination and completeness of the construction documents on constructability cannot be overstated.

Contractors use the design documents to develop and submit estimates and formal bids for construction of the project, and ultimately implement the design if their bid is selected. Thus, the accuracy of estimates and success and responsiveness of subsequent bids to the owner's requirements are dependent on coordination and completeness. The documents must describe in sufficient detail the elements of the project to be built, the scope of the work required, the applicable quality requirements, and any special or extraordinary requirements governing the construction. If the documents reflect a high level of completeness and coordination among the architectural, structural and mechanical drawings, the construction process should proceed smoothly from the initial design phase through construction.

In contrast, the trend is opposite. The variety of warp-speed delivery packages now available lead to many projects being prepared with unreasonable allowances for design time and budget and unrealistic expectations by the owner. This demand for expedited design services has led to the issuing of incomplete and uncoordinated construction drawings for bid. Sometimes, it seems to be the expectation that the contractors will complete and coordinate the designs, check for and find errors, and anticipate the costs of doing so in their bids. This is not the case, and furthermore, it is not in compliance with the standard of care of a professional engineer.

This practice leads to confusion among the bidders and bids that inevitably are based on erroneous assumptions. Ultimately, the result is a disservice to the owner and project participants, including the bidders, and may lead to future disputes, extra costs, late delivery and eventual litigation—anathemas to any proponent of constructability. An owner or project participant who chooses to pursue this unfortunate model should be made aware of the contingency funds that will likely be necessary to cover the cost of the eventual changes necessary to complete the project. [See Section 3.6, Fast-Track Project Delivery, in the AISC *Code of Standard Practice for Steel Buildings and Bridges*, hereafter referred to as the *Code of Standard Practice* (AISC, 2005c)].

Documents that are incomplete or uncoordinated tend to complicate or break down the construction process, which is the opposite of what constructability aims to achieve. Inaccurate bids or incomplete project proposals generally result in designs that must be completed through revisions, change orders, and numerous requests for information (RFIs), and conflicts among the design and construction team, especially about connection details. These conflicts, if not quickly resolved, often increase the "blame-game" while decreasing morale, productivity, construction quality, and the ability of the parties to make a reasonable profit. Project costs increase, schedules extend, misunderstandings abound, the owner is disappointed, and costly litigation ensues.

CASE 962D and the *Code of Standard Practice* provide clear guidance for what information should be included in construction documents for steel detailing, fabrication and erection. Yet what if the information the structural engineer needs from the architect is not yet available? What if the need for accurate dimensions for the structural construction documents, and even for the detailers to produce shop drawings, is not recognized? Perhaps the SER in this situation can convince the owner and design team that issuing incomplete construction documents to accelerate the start of construction may instead extend the schedule for final occupancy of the building, and ultimately increase total construction cost.

Coordination and completeness must be reviewed throughout the design process. It cannot be left to a final check at the end of the construction document phase. Each member of the design team should have, and follow, a quality management plan that includes written procedures for processing its work. The plan should include:

- A review of design decisions.
- Guidelines for the preparation and checking of calculations.
- A review of the transfer of information from calculations to BIM or drawings.
- A review of coordination with other disciplines, including "clash" checks.
- Guidelines for dimensional requirements or checks.
- Coordination of drawings, general notes and specifications.

- Procedures for checking configured to result in the confirmation and coordination of one's specific documents or models, and also the interrelationship of these documents or models with those of other disciplines on the project team.
- Confirmation and checking of schedules.
- Responsibility for checking.

Of particular note here, project specifications should be project specific and accurate in terms of the description of the construction, materials and processes required. Too often, specifications are prepared with a quick edit of an office standard specification, possibly leading to errors, incompatibility, irrelevant information, and needless expense.

Coordinated documents flow from a coordinated project team (design and construction), both of which are essential for a smooth-flowing, successful project. Some elements that must be considered when coordinating construction documents are:

- During the development of contracts for all of the design professionals, it is crucial that the scope of services for all team members be carefully coordinated so that each team member's responsibilities are clear and understood.
- The entire team must coordinate construction tolerances to allow for the integration of the various building systems to be used. Structural systems alone have a variety of applicable tolerances, including those defined in ASTM A6/A6M (ASTM, 2007), the *Code of Standard Practice*, ACI 117 (ACI, 2006), ACI 301 (ACI, 2005a), ACI 315 (ACI, 1999), ACI 318 (ACI, 2005b), Prestressed Concrete Institute documents, etc. Then there are the tolerances specified by individual product manufacturers, such as cladding, roofing, interior finishes and other systems. These tolerances must be considered and understood by all members of the design team.
- The foundation and superstructure design must conform to the design criteria presented in the geotechnical report.
- The location and magnitude of loads imposed by architectural, mechanical, electrical, plumbing and sprinkler systems must be addressed in the structural design.
- Beam penetrations or plenum spaces required for utilities must be known and accounted for in the design of the structure and structural floor system.

Coordination of documents goes well beyond checking that the structural gridline dimensions match the architectural dimensions and that the dimensions "close." Construction materials will always deviate from the ideal conditions shown on the drawings, and the design team must allow for the tolerance requirements of the various systems, some of which may not be inherently compatible. In extreme or unusual cases, it may be necessary for the design team to define special tolerances for systems or components and require all contractors to fabricate and erect its materials accordingly.

When specialty items are specified to be provided by a specialty structural engineer, the SER should be involved in preparing the specifications for the work to be performed, including design criteria, performance criteria and submittals. Particular attention must be given to ensure that the contracts for the SER and specialty structural engineer(s) have clear and compatible scopes of work, including the SER's review requirements and acceptance criteria. See CASE 962D and CASE "National Practice Guidelines for Specialty Structural Engineers" for further information (CASE, 2005).

2.3 PROJECT COMMUNICATION

Regardless of the project delivery method chosen by the owner, the owner's designated representative for design should establish a communications protocol for the project to document the formal lines of communication, the process for written confirmation, and a means to establish follow-up. The protocol should also establish systems to be followed and audit procedures to ensure that these systems are being followed.

A complete roster should be created and distributed showing all team members, including the name of each firm or company, appropriate contacts, addresses, telephone numbers, fax numbers, and e-mail addresses. If a networked or online system is to be used, access information for this should also be included.

Regular team discussions and clear and direct communications are vital. A schedule of meetings should be established and provided to all participants. Meetings should always have an agenda and meeting minutes should be distributed to all attendees and other appropriate parties. Meetings should be regularly scheduled with a frequency that is appropriate to maintain control of coordination and completeness. Key subcontractors should also be involved and communication protocols established with them as well.

Similar procedures should be followed internally by each design subconsultant, starting with an initial review for each team member. That review should include all engineers and drafters assigned to the project so that they also understand the goals of the project and how each can contribute to reaching those goals.

2.4 CONSTRUCTABILITY INPUT

Without experience, one does not know what one does not know. The problem with experience is that our education is enhanced more by our bad experiences than from our good experiences. With today's world changing so fast, it is impossible for anyone to know everything; hence, the benefits made possible with constructability input and involvement of steel construction expertise in the early project decisions. This input and infusion of knowledge is primarily for the owner's benefit, and the owner should be advised that the understanding and implementation of the principles of constructability by the design team will go a long way toward ensuring the goal of in-budget and on-time completion of the structural steel portion of the project.

2.5 DESIGN AND CONTRACTOR COORDINATION

The coordination begins with the architect, SER, and other design disciplines. The drawings must be coordinated among the disciplines at the time the bid packages are developed. The SER should review the nature of the structural steel design and its lateral-load-resisting system with the owner's designated representative for construction (ODRC) to allow the construction of the project to be properly planned and sequenced.

Time must be allocated for the ODRC to review the work of the previous trade. For example, in the steel erector's case, the ODRC surveys the column base plate anchor rods, locates the load bearing walls, and establishes the control lines. The next contractor can then move in, get the work in that sequence completed, and move to the next sequence. Each follow-on trade follows this same process.

Teamwork is an essential element of construction. The ideal process from a construction viewpoint will allow each trade to come to the jobsite when scheduled; perform the work in an efficient, orderly manner; and leave the jobsite never to return (except for the completion ceremony). This is an ideal concept that is attainable on most projects with proper planning.

The contract should clearly define who is responsible for design coordination and who is responsible for contractor coordination.

Chapter 3
The Design Process

The success of the design process is directly related to the ability of the design team to communicate and coordinate their activities. The design team must freely discuss expectations and requirements before the project starts. These discussions must include the division of responsibilities among the team members, the project milestones, and the expected deliverables at those milestones. These discussions should be recorded in writing, and the decisions reached should become a part of the contract between the team members.

Design team members should use their education and experience to translate architectural concepts into a defined structure while evaluating material selection, constructability, cost and schedule. The team should attempt to maximize shop production and minimize field labor, thereby enhancing the entire construction process. A set of contract documents prepared by such a team will be complete and coordinated.

Structural engineers may or may not be familiar with industry standards, material availability, fabrication processes and erection standards. Therefore, when making decisions that define the structure, they should seek the most up-to-date and complete information available in the industry.

3.1 DISCUSSION TOPICS

As the design stage of a project begins, general structural design topics such as the following should be discussed by the design team with input from the fabricator, erector, construction manager and/or others:

- How can repetition of beam and column sizes—even at the expense of some added member weight—be used to reduce total project cost? Repetition simplifies detailing, fabrication, and erection, reducing piece marks and handling costs; provides for ease of erection; and minimizes field handling, storage, detailing and modifications. This same idea applies also to uniform selection of bay sizes and the orientation of columns.

- At what interval should column splices be located? Maximizing the practical column length will minimize the number of column shafts. The practical limits are determined by type of building, shipping restrictions, total weight of column, erection sequence, or, in some cases, by union contract.

- Are all structural steel bases located on footings? If any steel is to be erected on secondary pours, such as foundation walls, can these details be modified?

- Is it possible to separate masonry support steel from structural steel? If the building facade is masonry, consider making the two systems independent, as this simplifies detailing, structural steel erection, and masonry installation.

- Must materials be mixed or can mixing of materials be eliminated? Avoid mixing materials (e.g., structural steel, load-bearing masonry, precast concrete) in the primary framing. The coordination required among trades can add considerable time and cost to a project. If the final design requires the mixing of materials, define the lateral load resisting system and supporting diaphragms, provide details of the required interface, and provide the information required in the *Code of Standard Practice,* Section 7.10, Temporary Support of Structural Steel Frames (AISC, 2005c).

- What tolerance issues need to be considered in the design and construction of the project? Have the concrete contractor and steel contractor coordinated the tolerances that will apply at column base locations? The *Code of Standard Practice* requires more stringent placement tolerances for anchor rods than ACI 117 (ACI, 2006) allows for concrete industry embeds. The contract documents can simply require that *Code of Standard Practice* tolerances be met for anchor rods in the concrete.

- How can the amount of loose material be minimized? Loose material is difficult to track, easily lost, and usually requires the steel contractor to return to the site or erect out of sequence.

- What connection types make the most sense for the project? As an example for shear connection, single-plate or single-angle shear connections may be preferable because they involve fewer detail pieces and allow for fast and safe erection.

- How should member sizes be modified to facilitate uniformity, practicality in making connections, and ease of installation?

- Where can built-up shapes and special details be eliminated in favor of common details and standard hot-rolled shapes?

- AISC standard connection details can often facilitate shop drawing preparation and review, and provide connections that are familiar and easy to fabricate and install.

- How can the number of different bolt sizes and grades required on the project be minimized?

- Do the connection details ensure that it is physically possible to install the bolts in their specified locations?

- Where is it advisable to use short-slotted holes, long-slotted holes, or oversized holes as applicable for fit-up?

- Are snug-tightened joints permissible for the project? If not, where are pretensioned or slip-critical joints required? Slip-critical joints are only required in specific cases as listed in the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-05), hereafter referred to as the AISC *Specification* (AISC, 2005a) and RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, hereafter referred to as the RCSC *Specification* (RCSC, 2004).

- Is it possible to eliminate stiffeners and/or web doubler plates to reduce total project cost? These items are very costly due to the extra detailing, number of pieces to fabricate, fit-up, and welding and the accompanying inspection requirements. AISC Design Guide No. 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999) provides guidance on this issue.

- Are adequate clearances provided for welding? Joint quality, as well as weld quality, may suffer if adequate welding access is not provided. This may seem to be a fabricator problem; however, if the detail is too difficult to fit-up and/or weld, the integrity of the entire structure may be compromised. Such details are best taken care of during the design process by the structural engineer of record (SER).

- Is camber required? If so, how has the designer calculated it and what is the goal of the cambering and what variation has been anticipated?

- Is super-elevation required? If super-elevation (camber in two directions) is required, the SER must specify in the contract documents the location and magnitude of the super-elevation, explain the nature of the super-elevation, note the acceptable tolerances.

- For SER-designed connections, is the design complete? This includes such information as the complete connection detail, including material thicknesses, welding and/or bolting requirements, and stiffeners and/or doublers designed. Alternatively, is sufficient information provided for connections that are to be completed or selected by an engineer working for the steel fabricator? It is essential that the connection requirements and acceptance criteria be established by the SER on the design documents. See the *Code of Standard Practice*, Section 3.1.2.

3.2 JOINT DETAILS

It is often helpful, and sometimes critical, to discuss layouts and geometry for nontypical details, especially for multi-member joints and joints that transfer loads between structural steel elements and nonstructural steel elements. The challenges might not be evident in the schematic joint shown in Figure 3-1 (note transfer of force into the concrete shear wall), but likely will be identifiable when viewed as given in Figure 3-2.

Figure 3-1 depicts a partial fifth floor plan as shown on the original design documents. Upon award, the fabricator retained a specialty structural engineer (SSE) to lay out and design the nontypical connections utilizing the information and loadings provided on the original design documents. Upon developing the layout for this joint, resolving forces, and designing the connections of the steel members to the column, the SSE discovered that the mechanism needed to

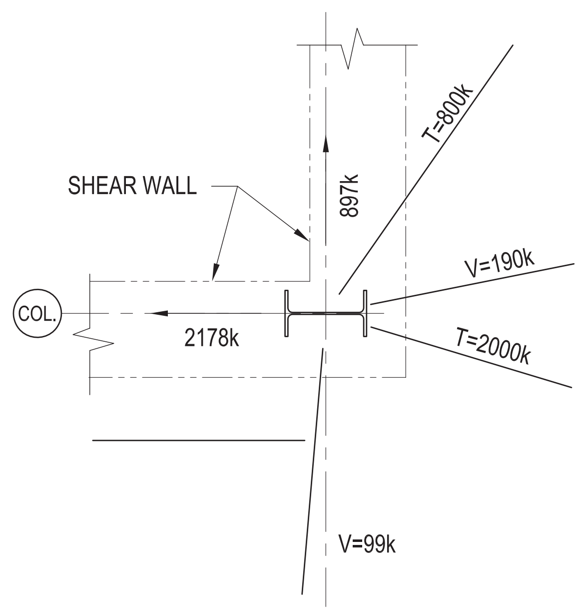

Fig. 3-1. Schematic diagram of joint shown in detail in Figure 3.2.

transfer these forces through the column into the concrete shear wall did not exist anywhere in the original design documents. Subsequent design by the SSE generated the joint shown in Figure 3-2.

It is examples such as this one that illustrate how valuable constructability discussions can be. In fact, without such a discussion, it is much more difficult to ensure that the details provided match the design intent.

3.3 INTERDEPENDENCE

The design team should seek help to identify the interdependence of each construction activity and then assist in developing a means to reduce the interdependence and make each activity as independent as possible. The purpose is to simplify the construction process by eliminating or reducing the necessary coordination of trades.

Each project has its own requirements and constraints; therefore, there are no pat answers. The goal is to minimize the interdependence of trades, allowing the structural steel fabricator to complete the shop details without delay; eliminate any need to modify the fabrication once underway; and, when the structural steel reaches the field, reduce unnecessary job site coordination.

This can be accomplished by discussing the following architectural coordination topics early in the design stages of a project:

- What is the interdependence of systems and construction activities? Can the systems and activities be made more independent? The construction process is generally simplified by eliminating or reducing the necessary coordination of trades.

- Is the curtain wall system free of special holes or connection elements that must be affixed to the steel frame by the steel contractor? If not, has the necessary information been clearly defined within the contract documents?

- Does the curtain wall anchorage system have sufficient adjustment to accommodate the steel frame tolerances and curtain wall installation tolerances? Are the tolerances compatible?

- When will the locations of horizontal runs and vertical risers, stair openings and elevator shafts, and similar features be established?

- For crane runway structures, establish the crane clearance envelope, crane class, crane loading, and service limits prior to determining the class of building and developing the framing options.

- When will roof-mounted equipment and roof penetrations be located and sized? If this timing is an issue relative to the timing of steel fabrication and erection, consider using a design that allows for field assembly and installation. For example, Figure 3-3 shows a module that is designed to be placed anywhere within a bay. In this detail, the A and B dimensions should be maximized to the largest possible opening requirements, and then flashed to the desired size.

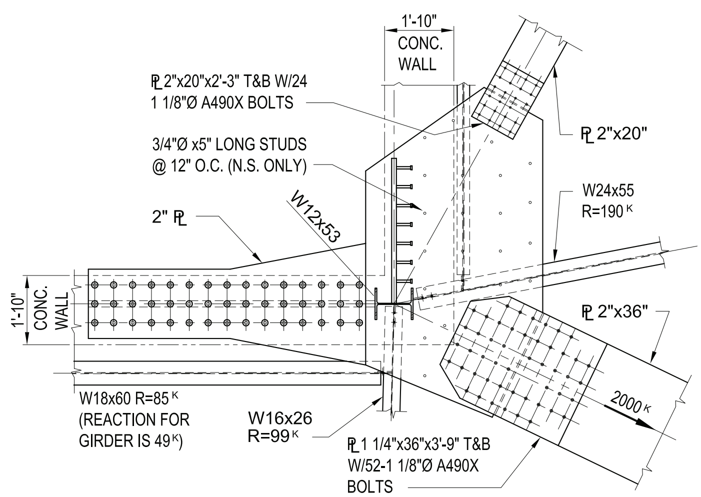

Fig. 3-2. SSE-generated connection layout (shown schematically in Figure 3.1).

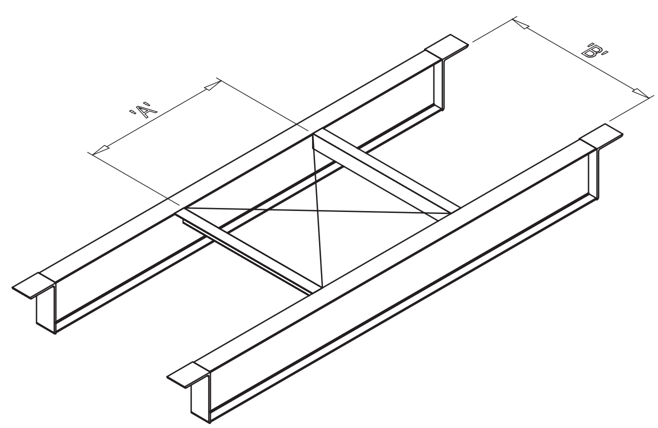

Fig. 3-3. Mechanical roof penetration framing module.

Presumably the foregoing discussion topics will spark discussion of other applicable topics between the owner, design team and construction team. All the better!

3.4 CODE OF STANDARD PRACTICE

The preface of the *Code of Standard Practice* states:

> This Code provides a useful framework for a common understanding of the acceptable standards when contracting for structural steel. As such, it is useful for owners, architects, engineers, general contractors, construction managers, fabricators, steel detailers, erectors and others that are associated with construction in structural steel. Unless specific provisions to the contrary are contained in the contract documents, the existing trade practices that are contained herein are considered to be the standard custom and usage of the industry and thereby incorporated into the relationships between the parties to a contract.

The SER, as the design team member responsible for the structural design and with the most direct contact and knowledge of the *Code of Standard Practice*, is the most likely team member to keep the other design team members abreast of the standards and customs of the steel industry. It is crucial for the design team to understand what is (and is not) contained within the *Code of Standard Practice* because, unless specified otherwise, the trade practices outlined in that document are provided.

Chapter 4
Structural Steel Framing

Section 2.1 of the *Code of Standard Practice*, defines what elements are "structural steel," and Section 2.2 lists those steel and metal items that are not. The distinction is important when assigning contractual responsibility for furnishing structural steel and other elements.

The provisions of the *Code of Standard Practice* are not intended to apply to items in Section 2.2. Examples of non-structural steel items are metal deck, open-web steel joists, joist girders, field-applied shear studs, and permanent suspension cables. Instead, documents from other organizations, like the Steel Deck Institute (SDI) and Steel Joist Institute (SJI) govern.

When such items are an essential part of the structural design, tolerances and installation requirements must be coordinated with the structural steel and clearly identified within the construction documents. When such items are contracted to be provided by the fabricator, coordination will be required between the fabricator and other material suppliers and trades.

4.1 ESTIMATING THE COSTS OF STEEL FRAMING

Maximizing economy is one aspect of constructability. This objective will not be attained if the owner is not furnished at the outset with realistic cost estimates for steel framing and other building components. The best way to estimate the cost of steel framing is for the project estimator to contact several structural steel fabricators. They are in constant contact with shapes producers and steel service centers, and are in the best position to assess the current market costs and availability of raw materials, including the potential for fluctuations. Of course, there are factors other than material costs that make up the steel frame cost estimate, such as:

- The coordination and completeness of the construction documents.
- The site conditions and special aspects of the project.
- The efficiency of the framing scheme and economy of the associated connections.
- Domestic versus imported shapes.
- Number of pieces or subassemblies to handle and install.
- Bolts—snug-tightened versus pretensioned.
- Field welded versus field-bolted construction.
- Fabrication considerations, such as for built-up girders or columns, shop assembled or knocked-down trusses, camber, and coating requirements.
- Special installation considerations, such as for shoring, jacking, super-elevation, temporary works, etc., if specified.
- The local or regional demand and capacity of fabrication and erection at the time of bidding.
- Whether material will be sourced from shapes producers directly or from service centers.

Many of these factors can be quantified during the planning stage, and those that cannot can be foreseen and addressed through contingencies in the project cost estimate.

One continuously occurring enigma in estimating is the construction manager's perception that economy in structural steel construction can be measured in dollars per ton or pounds per square foot. The dollars-per-ton version of this myth is probably caused by bids given in the form of dollars per ton (after detailed take-offs to arrive at total construction cost, of course). Yet, all in the industry—and most who apply the foregoing enigma—know that this myth almost never is true. The cost of the steel frame, fabricated and erected, is partly under the control of the architect and structural engineer of record (SER) in the form of framing efficiency, selection of the lateral load resisting system, and connection design. But the real costs as estimated by fabricators and erectors are not proportional to weight. Yet, it remains the tendency to look only at total steel weight to assess economy.

In reality, dollars per square foot is the measure of the cost that matters to the owner. That is, a steel frame heavier than an arbitrary weight threshold may indeed have a lower total cost. Many an owner has paid too much because a weight-conscious estimator discarded the real economy in favor of a least weight solution, thus negating the creative thinking of design professionals and steel contractors.

4.2 AVAILABILITY OF STEEL SHAPES

As with estimating the cost of structural steel framing, the best way to determine availability and lead times for steel framing is for the project estimator to contact several structural steel fabricators. Again, they are in constant contact with shapes producers and steel service centers and are in the best position to assess the current availability of raw materials, procurement constraints, and fabrication times. Fabricators can also provide guidance on what material grades, shapes and size ranges are (or are not) procurable in the timeline of the project.

Three indicators must be considered when determining the on-site delivery schedule for structural steel:

- Are the specified structural materials produced and readily available?
- Where are the materials produced? How available are the materials?
- When will these materials be delivered to the fabricator?
- When will the fabricator deliver the fabricated product to the site?

A fabricator is best able to answer each of these questions, and the answers can make all the difference. Instead of time-consuming changes and rework that might otherwise occur, the design team can take advantage of current availability information. This will likely yield shorter lead times, more economical pricing, improved delivery times, and an erection schedule that meets or exceeds the owner's expectations—all while minimizing overall project costs.

Timely communication with fabricators can facilitate the placement of early orders for the structural steel package, which is a key means to accelerate the on-site delivery of structural steel. Communication and an early order can also help alleviate concerns with fluctuations in material prices.

Both shape availability and appropriate grade specifications are more dynamic than static. Several years ago, ASTM A992 replaced ASTM A36 and ASTM A572 Grade 50 for applications using W shapes. ASTM A36 material is still commonly used for plates, angles and channels. Hollow structural sections (HSS) are primarily furnished as ASTM A500 Grade B. Pipe is primarily furnished as ASTM A53 Grade B. Appropriate material specifications are listed in Part 2 of the AISC *Steel Construction Manual* (AISC, 2005d), and this list is periodically reviewed and updated in *Modern Steel Construction* magazine. The most current revision of this feature is available at www.aisc.org/steelavailability.

For major structural shape availability, AISC publishes an annual summary, both in *Modern Steel Construction* magazine (W shapes are summarized in each January issue; HSS are summarized in each July issue) and on its Web site at www.aisc.org/steelavailability. Contact information is also provided for steel shape and HSS producers, which in most cases have links to Web sites with more detailed information on current pricing and rolling schedules. A cautionary note: shape availability changes as higher-demand shapes are rolled more often than lower-demand infrequently used shapes. These shapes may be listed in the summaries of all shapes, but it may only be available by special order or quantity. Examples of this include large rectangular HSS and heavy W shapes. This emphasizes the benefit of discussions with fabricators early in the project.

4.3 MILLS AND SERVICE CENTERS

Mrozowski (1999) states that "mills often require that steel members be purchased in 5-ton bundles with a minimum order of 20 tons. This may be a problem when only a small number of certain size members are needed, or if a member is a less common size." Historically, this issue has been managed with the use of repetitive members in the design, early mill orders, and fabricator inventories. Material shortages, mill rolling schedules, and delivery schedules could still cause delays, however.

In recent years, mill quantity thresholds have reduced significantly and the steel service center (also known as a "warehouse") has become vital to the steel material supply chain, even for larger projects, in solving such concerns. More than half of the structural shapes produced in the United States are sold to steel service centers, not to fabricators directly, which serves to modulate the effects of material shortages and rolling schedules on availability. Steel service centers also help reduce the inventory a fabricator must keep on hand. Although this service comes with a price premium for the material, steel is more readily available and can often be delivered to the shop in days rather than weeks or months from producers.

4.4 SUSTAINABLE DESIGN

If the building is to be LEED Certified, the selection of structural materials and component design may be affected. LEED stands for Leadership in Energy and Environmental Design. The decision to design for a LEED certification should be made at the earliest planning stage, as the LEED rating system relies on the project team to generate proof of compliance. AISC has published several articles on how to assess the value of steel as a construction material under the LEED program, and these are available at www.aisc.org/sustainability.

Each structural system has opportunities and constraints when evaluated as a part of an environmental or "green" design effort. Market demands for steel production spur a significant amount of recycling, inherently contributing to sustainable design efforts. According to the Steel Recycling

Institute (2002), 67 million tons of steel were recycled in the U.S. alone in 2001. Worldwide, 400 million tons of steel were recycled—one and a half times the amount of all other recycled materials combined, including paper, glass, aluminum and plastic. Approximately 40 million tons (59% of total recycled steel) were derived from construction and demolition waste, and the steel salvage market accounts for an additional 4 million tons per year. Each ton of recycled steel saves 2,500 lb of iron ore, 1,400 lb of coal, and 120 lb of limestone. In addition, recycling requires less energy, creates less waste, and releases less pollutants than producing the same amount of steel from virgin materials. Recycling, however, is only one aspect of how structural steel can contribute to green design efforts, and the steel frame is only one component of the overall structural system.

The LEED rating system is designed for new and existing commercial, institutional, industrial and multi-story residential buildings (Eckmann, Harrison, Ekman and Stern, 2003). Its purpose is to set an industry standard for green buildings and, in doing so, help drive the marketplace toward more sustainable development. It provides an accessible and understandable framework, and a recognized reference for project teams to make decisions and evaluate the overall performance of a sustainable building design effort. LEED was developed by consensus of the membership of the U.S. Green Building Council (USGBC), which includes companies from all segments of the building industry—a membership that has grown exponentially since 1998. LEED is the most widely used green building rating system in the U.S. The certification process requires the project team to pursue and evaluate specific credits, to document requirements successfully met for each credit, and to submit credit documentation to the USGBC for review (see www.aisc.org/sustainability). Because the LEED rating system relies on the project team to generate proof of compliance, LEED is considered a self-certification system.

4.5 SYSTEM AND MEMBER SELECTION

Selection and sizing of steel framing systems and members is primarily based on the forces, deformations and other effects for which the systems and members must be designed. Constructability concepts, when applied to the steel framing, can decrease detailing, fabrication, erection, and inspection time and cost. These concepts include the use of repetitive beam and girder sizes, optimizing column splice locations, upsizing columns to minimize reinforcement, utilizing snug-tight bolts, and resizing floor beams for wider spacing.

The selection of steel framing systems often involves consideration of many different factors. Constructability considerations vary across the country due to the local labor pool, skill sets, and local practice. Building geometry, bay spacing, lateral load resisting systems, framing options, floor systems, roof systems, and serviceability are all important considerations in constructability. The following general guidance can be used as the starting point for constructability discussions that are specific to the project. For further information, AISC Design Guide No. 5, *Design of Low- and Medium-Rise Steel Buildings* (Allison, 1991) is an excellent resource on selecting steel framing systems.

4.5.1 Floor and Roof Framing

For floor framing, Ruddy (1983) states that the use of W-shape girders tends to maximize economy with a bay length of 1.25 to 1.5 times the width, a bay area from 750 to 1,250 ft^2, and the infill beams or steel joists spanning in the long direction. For roof framing, these recommendations also apply; additionally, roofs sometimes are configured with girders that cantilever over the tops of the columns (Rongoe, 1996). AISC Design Guide No. 5, *Low- and Medium-Rise Steel Buildings* (Allison, 1991) includes a study based on data from two fabricators from two different geographic regions, which also supports the above recommendations. In addition, Ericksen (2005) notes that the major considerations when comparing floor framing options are the level of composite action, whether or not to camber, the bay dimensions, the beam spacing, and the depth of the floor framing.

Larger bays and/or greater beam spacing (e.g., see Figures 4-1 and 4-2) may mean deeper and heavier beams, but there are also fewer components and connections to detail, fabricate, erect and inspect, as well as fewer foundations to design and install. The cost of detailing, fabrication, erection and inspection for a small beam is essentially the same as

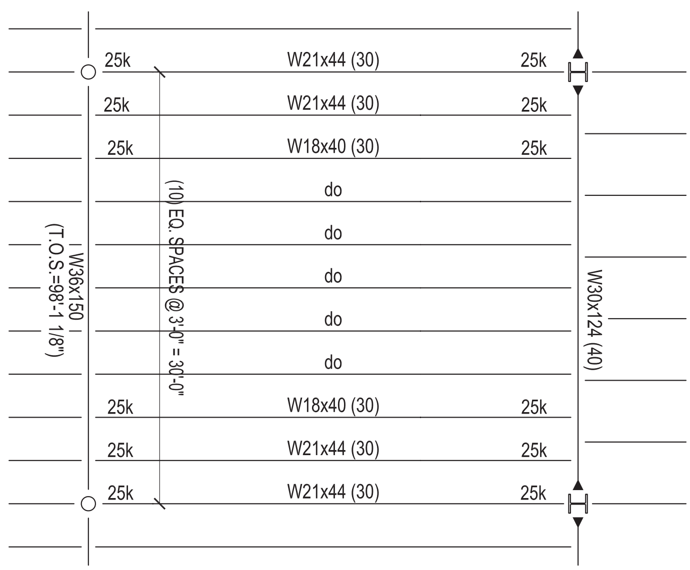

Fig. 4-1. An over-framed bay.

for a large beam, whereas the savings involved in reducing member weight is primarily savings in the cost of mill material. Thus, when the number of pieces is reduced, the costs of detailing, fabrication, erection and inspection are reduced at a far faster rate than the additional cost of steel. See also Case Study One at the end of this chapter for an example of this concept.

Composite action can significantly reduce the weight and/or depth of the steel floor framing. However, full composite design is often not necessary and its benefit must be compared with the cost of installed studs, and the construction time required coordinating and installing them. Noncomposite construction should be considered when there are few members that benefit from composite action on the project, as the costs to mobilize an installation crew and the added costs associated with small quantities of studs may exceed the benefits associated with composite design.

Ultimately, roof and floor system design is more than determining the strength of the supporting structural members. The selection criteria might differ drastically depending upon which of the following (or other) criteria apply:

- Strength versus serviceability requirements.
- Composite versus noncomposite.
- Serviceability considerations, such as dead load deflection criteria and camber, live load deflection criteria, floor vibration criteria, etc.
- Shape selection for uniformity and/or repetition of size.

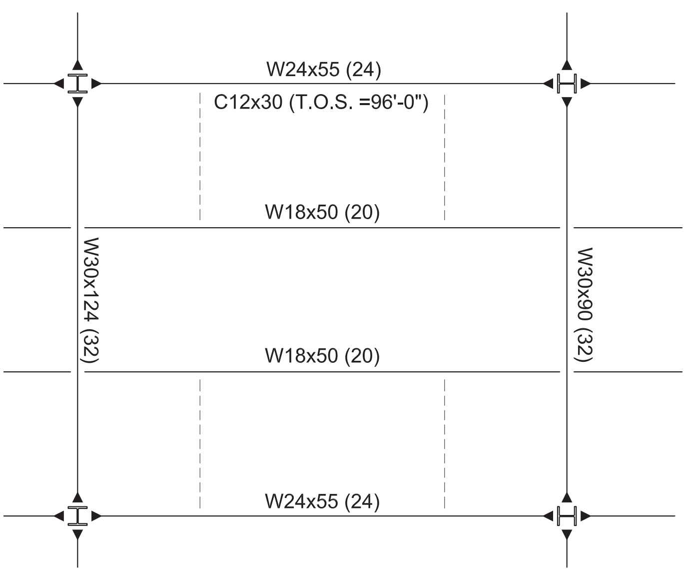

Fig. 4-2. An efficient bay framing arrangement.

- Connection considerations—a deeper girder might be used to simplify the connection of a supported beam, or to allow the infill beams to be lowered to eliminate coping of infill beams.
- Construction considerations—members that are too limber are difficult to install and can create floor deflections that are unacceptable to the owner or even unsafe. The following rules of thumb for minimum depth-to-span ratios may be helpful:

  Beams for typical framing 1:24
  Beams subject to vibrating activities or
  equipment 1:16
  Girts 1:50
  Purlins 1:32

  Note that these ratios are recommendations, not requirements.
- Occupational Safety and Health Administration (OSHA) considerations—the impact of OSHA regulations, such as those required for stability of joists during erection, must be considered.

Constructability will also be improved with attention to other details in the design stage, including:

- Occupancy and loading requirements.
- Serviceability requirements, including deflection, vibration, sound transfer and insulation.
- Fire rating requirements.
- Requirements specific to the final finishes.
- Metal deck type, depth and gage, as well as whether the deck will be plain, painted or galvanized.
- Diaphragm requirements and attachment details.
- Potential for conflicts with framing details, such as interference at beam-to-column moment connections where the top flange connection bolts may require a detail with the deck cut to clear the bolts and supported on added angles or plates.
- Slab edge form requirements. Bassar (2002) provides a summary of common concerns and suggestions for a practical approach to the design and installation of edge form. Note that field installation is recommended for slab edges because normal construction tolerances generally cannot be accommodated with shop attachment.

4.5.2 Long-Span Framing

Long-span framing is usually accommodated with trusses. The following rules of thumb for minimum depth-to-span ratios may be helpful:

Roof truss > 1:12
Floor truss > 1:10
Bridge truss > 1:5 to 1:10

Note that these ratios are recommendations, not requirements.

A Warren truss generally will weigh less than a Pratt truss, but least weight is not always least cost. The truss configuration should be selected with consideration of span, usage, depth restrictions, appearance requirements, connection requirements, details for intersecting members, and truss constructability considerations, such as:

- Splice locations.
- Member weight.
- Lateral stability.
- Temporary construction loading.

Panel points are often selected based upon bay width, truss geometry, and consideration of concentrated forces on the truss chords. Note that long panels will increase the weight of the truss, while shorter panels will increase the fabrication cost of the truss. As a rule of thumb to find the right balance between these factors, the diagonal web members should fall in the range of 45 to 55° from the horizontal. The chords may consist of angles, W shapes, WT shapes, or HSS or pipes. Each shape has its benefits:

- Angles are often the least weight solution, but not often the least cost solution because they require stem plates for accepting web members.
- WT shapes are an option in lieu of angles where the stem can be used to connect the web members. As loads increase, the end panels may require web extensions to handle the shear forces. Note that WT shapes are split from W shapes and must be straightened after they have been split, adding cost.
- W shapes may be used with the web vertical or horizontal. They are quite strong and stiff and provide stability for long spans during erection. The use of W shapes can reduce the depth required for the truss, although this will tend to increase the forces in all of the members.
- HSS are primarily used in areas where aesthetics is a priority. The connection of HSS web members is normally more costly than the former options.

Similarly, for web members:

- Angles are the shape of choice for most industrial trusses. They are rolled in a wide variety of sizes and thicknesses, generally available throughout the country, and easily fabricated.
- WT shapes can be used in lieu of angles at the designer's discretion. The only advantage may be when larger sizes are required.
- W shapes are usually used only when necessary for extreme loads or long spans, such as arena and auditorium roofs. When W-shape web members are used, consideration should be given to using W-shape chords with the webs horizontal to allow flange-to-flange web member connections.
- HSS can be used with W-shape chords as well as HSS chords. The ends of HSS can be slotted to accept gusset plates for attachment to W shapes, WT shapes or angle chords. Or, HSS can be cut to fit the surface of the HSS or W-shape chords and welded with fillet welds or partial-joint-penetration or complete-joint-penetration groove welds.

4.5.3 Columns

A column can be a W-shape; hollow structural section (HSS), which can be round, square, or rectangular; or a cross-section built up from shapes or plates. W-shape columns may be a better choice in multi-tier framing, while HSS columns may offer an advantage in single-tier projects. Moment connections, when required, are more straightforward with W-shape columns. Nonetheless, the specifics of the project should be evaluated when deciding which option offers the most desirable benefits.

The SER's initial column selection should be primarily based on:

- Current cost.
- Availability.
- Flange thickness—punched or drilled.
- Flange width—connection to other members.
- Width between flanges—access for connection to web.
- Architectural constraints.

On single-story structures with the beam framing over the columns, the size of the column may be a W6 through W24, or larger. The majority of the columns for low- to mid-rise buildings will range in size from a W8 to W14. However, on taller structures, the columns must accommodate floor and roof framing. In most cases the W14 shapes provide the best alternative for columns, with W10 and W12 shapes closely behind based on the floor beam shapes and connection requirements. HSS are highly efficient but do require the SER and owner to consider connection detail requirements and relative material and fabrication costs, respectively.

4.5.4 Braced Frames Versus Moment Frames

HSS, W shapes, angles, channels and structural tees can all be designed in various configurations for bracing to provide lateral load resistance and stability in the completed structure. The choice of brace configuration (V, inverted-V, X, eccentric, etc.) may be influenced by architectural requirements, building geometry, and height and connection requirements.

Historically, moment frames have been favored by architects for the increased flexibility due to elimination of diagonals, but usually at some cost penalty over braced frames. It should be noted, however, that costs of both braced frames and moment frames are highly dependent on the complexity of the corresponding connections required to transfer the lateral forces through the framing system. Constructability discussions should explore the details, as well as the associated costs, so that decisions can be made with accurate and appropriate information.

4.5.5 Horizontal Bracing (Diaphragms)

The majority of building floor systems utilize concrete on metal deck or metal deck diaphragms, but industrial buildings may not have floors or the roof system may require horizontal bracing. In such cases, there is an alternative to horizontal X-bracing (see Figure 4-3) for open framing and bottom chord bracing that may enhance constructability and reduce the cost of the bracing system. This alternative is called "diamond-bracing" as shown in Figure 4-4.

The diamond-bracing scheme is quite simple to design, detail and install. The shop connections can be welded or bolted as required by the steel contractor. The brace can be installed on top of the gusset plate, and the field bolted connections can be made without interference with other members (Figures 4-5 and 4-6). The details shown in Figures 4-7 and 4-8 illustrate the details for a horizontal bracing system that attaches to the bottom flange of the bottom chord. Case Study Two at the end of this chapter provides additional detail on this concept.

4.5.6 Vertical Bracing

There are many options for configurations of vertical bracing (Figure 4-9). In a steel structure, the most efficient system of bracing (optimum use of materials) is one in which bending is kept to a minimum. The V-brace or chevron brace, in which the horizontal element is supported (in high seismic regions, the intermediate support must be ignored when designing the floor beam) at midspan between columns, is more efficient than the X-braced system for several reasons.

In the X-braced system, the total length of the bracing member is longer. The floor beam spans full length and must support larger bending moments, resulting in large connections at the column. The system is not efficient as a tension/compression system. The chevron brace also offers

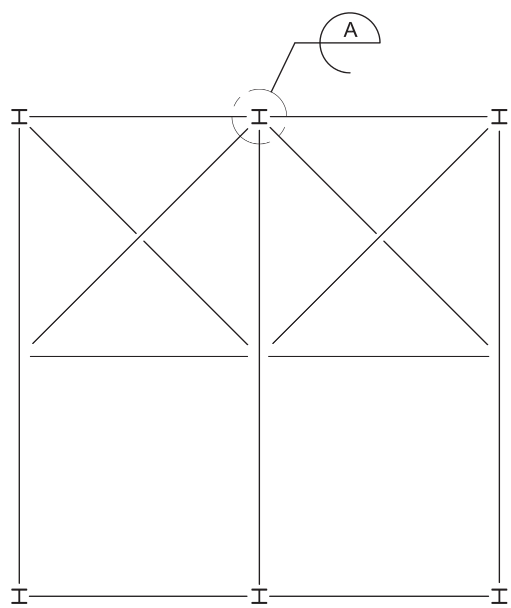

Fig. 4-3. Typical bottom chord X-bracing.

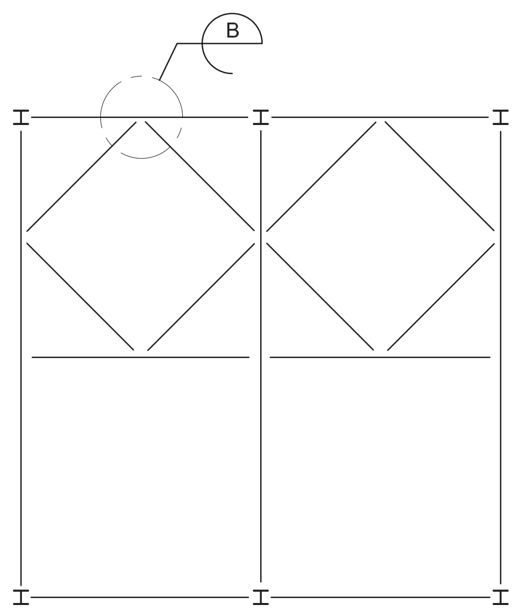

Fig. 4-4. Diamond bottom chord bracing.

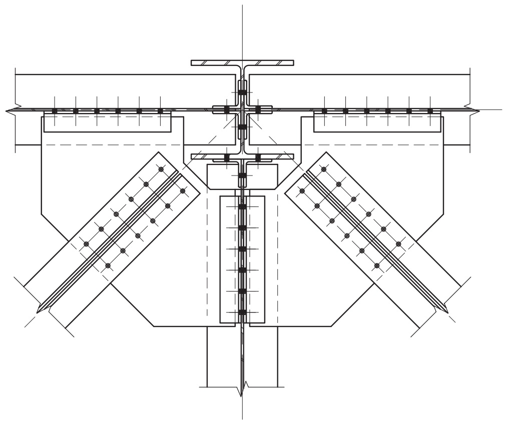

Fig. 4-5. Typical X-bracing detail at web.

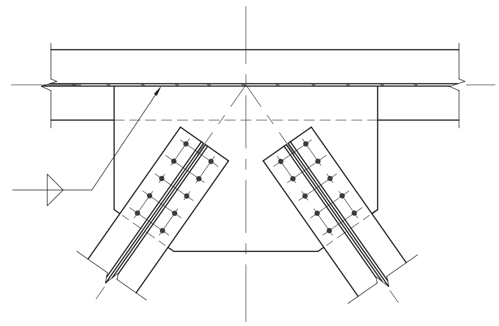

Fig. 4-6. Typical diamond bracing detail at web.

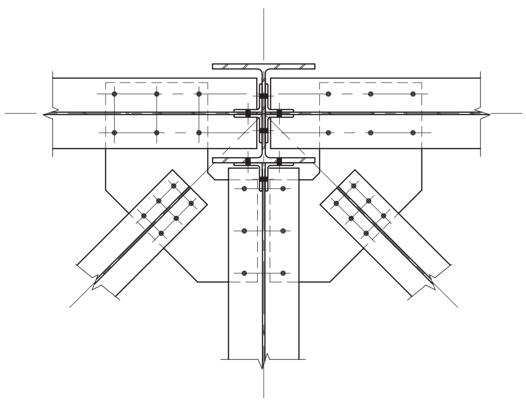

Fig. 4-7. Typical X-bracing detail at bottom flange.

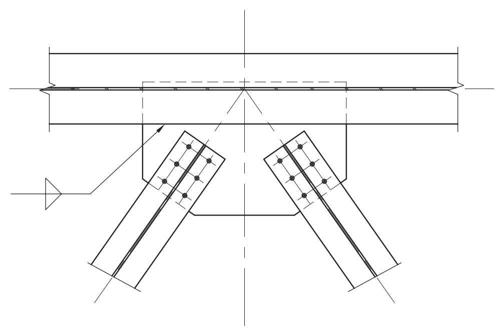

Fig. 4-8. Typical diamond bracing detail at bottom flange.

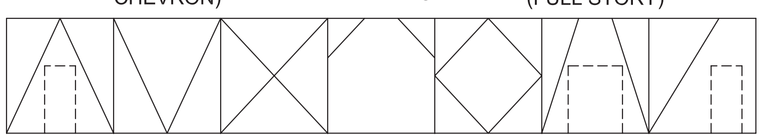

Fig. 4-9. Vertical bracing options.

greater freedom in the use of aisle space, since it is possible to fit doors beneath the apex. If still more space is required, the braces may be moved apart, resulting in a full-story, knee-braced bent. This induces significant bending moments into the column and floor beam, reducing the efficiency of the bracing system.

Chevron bracing, as noted in the *Modern Steel Construction* article (Marstellar, Mueller and Hewitt, 2002), "Chevron Bracing in Low-rise Buildings," is a common configuration for providing lateral-load resistance in low-rise steel framed buildings The article provides the basis for quickly estimating the brace size and connection material required for a given force designed to resist lateral forces due to gravity, wind and low-seismic loads (low-seismic loads are those where the seismic response modification factor, *R*, is equal to or less than 3 and no special seismic detailing is required). Chevron bracing elevations are shown in Figure 4-10.

The knee-braced scheme must be designed as a moment frame system which may not allow it to serve as a reasonable alternative.

The eccentrically braced frame is often used where larger openings are required and the floor beams are able to sustain the increased bending moments without significant increase in size and/or weight. There are many references which provide design and construction information within AISC technical journals and publications.

4.6 MINIMIZING FLOOR-TO-FLOOR HEIGHT

In many areas, total building height is restricted by local codes such that minimizing story height (i.e., total depth of floor construction) becomes a dominating design parameter. Fortunately, the designer has several options with steel framing systems. Conventional steel framing has been used in buildings with floor-to-floor heights as low as 8 ft 8 in. floor-to-floor by integrating the framing module within the walls and other architectural elements of the finished architecture (Millano, 2000; Kirmani, 2000). Additionally, there are special-purpose steel framing systems that can be considered, including staggered truss framing and floor systems that integrate mechanical systems within the depth of the structural system. At least one proprietary system is also available. These systems are discussed further below.

4.6.1 Staggered Truss Framing

Staggered truss structural steel framing utilizes story-high trusses (Figure 4-11) that span the entire width of the building with the trusses staggered from floor-to-floor. Often, the trusses support a precast hollow-core plank floor system—a light, dry, all-weather system, similar to steel. This combination allows for more column-free space, semi-finished floor and ceiling in one operation, lower foundation costs and seismic loads, and a rapid construction schedule. At least one study (Brazil, 2000) found that staggered truss steel construction has an economic and functional advantage over concrete flat-slab construction.

- Architectural configurations allow staggered trusses to be integrated into the partition walls between separate units at an interval that also allows an 8-in.-thick precast concrete plank floor. The resulting floor-to-floor heights can be as low as 8 ft 8 in.
- The framed space is column-free.

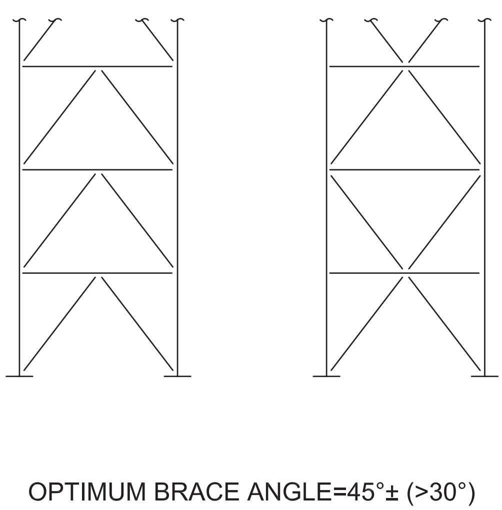

Fig. 4-10. Chevron bracing elevations.

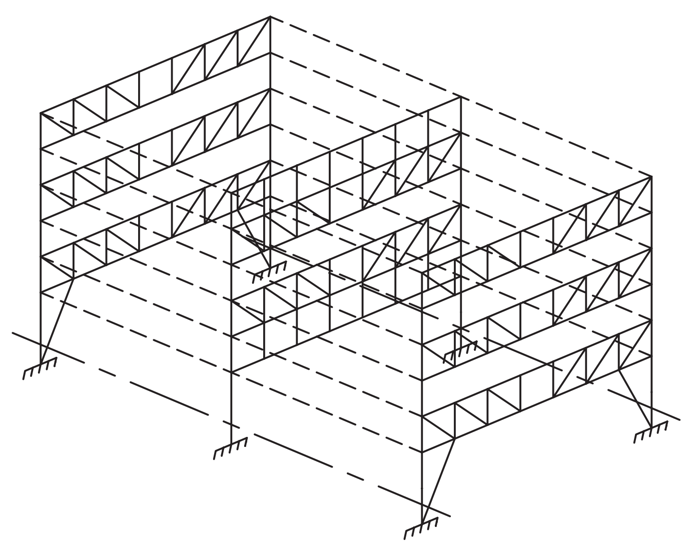

Fig. 4-11. Staggered truss system.

- The need for drop panels at columns is eliminated.
- The soffit of the planks can be finished in the same way as concrete slabs in flat-plate construction, without hung ceilings.
- The precast plank satisfies the floor system fire rating requirements.
- The weight of the steel and plank structure is 15% less than that of an equivalent concrete frame, which reduces foundation cost and seismic loads.
- The structure can be erected and enclosed faster than a concrete structure, allowing for lower construction loan interest cost and earlier occupancy.
- The total cost of the in-place structure is 10 to 25% less than a concrete-framed structure.

In addition to the above, when high-occupancy spaces such as ballrooms and theatres require clear spans of 40 to 60 ft, these can be accommodated with composite steel framing with slab on metal deck in place of the precast plank. In a concrete structure, transfer girders would be required to achieve such longer, more open spans. See AISC Design Guide No. 14, *Staggered Truss Framing Systems* (Wexler and Lin, 2002) for further information.

4.6.2 Integrated Structure and Mechanical Systems

Several floor systems can be used to integrate mechanical systems within the depth of the structural system, including beams with web penetrations, trusses, and steel joist framing. Often, these elements can be made composite with the concrete slab, using headed shear studs welded through the metal deck, thereby reducing the depth required. The openings provide a route for mechanical, heating, ventilating, plumbing and electrical systems.

Beams with web openings or truss framing may be preferable to joist framing, particularly for floor systems. Some steel joist series are not as flexible or easily reinforced for future increases in floor loading, or for concentrated loading.

4.6.3 Proprietary Systems

Proprietary systems such as the Girder-Slab™ system (Figure 4-12) also exist. This system uses a special fabricated girder to support precast hollow core planks that are then grouted after assembly, thus offering the advantage of a system that is like flat-plate concrete framing but entirely composed of pre-fabricated superstructure elements. The suitability, application, and use of the Girder-Slab™ system requires consideration by a registered design professional. For more information, see www.girder-slab.com.

4.7 CASE STUDY ONE

During design, an initial design concept for a 100,000 ft^2 manufacturing facility was discussed in a meeting between the design and construction team. The initial estimates were several hundred thousand dollars over the owner's construction budget, which would cause the project to be canceled. The structure consisted of roof trusses at 25-ft spacing supporting roof joists and three 20-ton underhung craneways. Perimeter columns and foundations were also at 25-ft spacing. The major framing of the initial design is shown in Figure 4-13 (girts and roof joist are not shown).

An alternative framing concept, as shown in Figure 4-14, was suggested after the discussion. First, the truss spacings were modified. Roof truss spacing was increased to 50 ft,

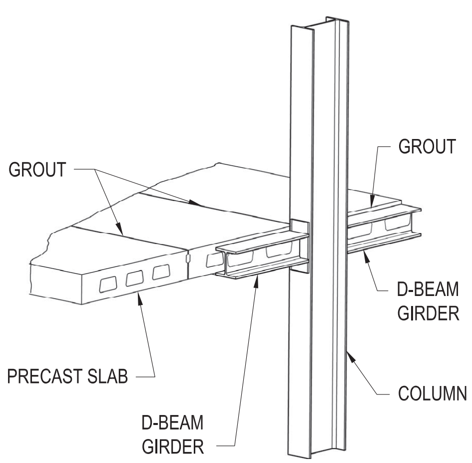

Fig. 4-12. Girder-Slab™ system.

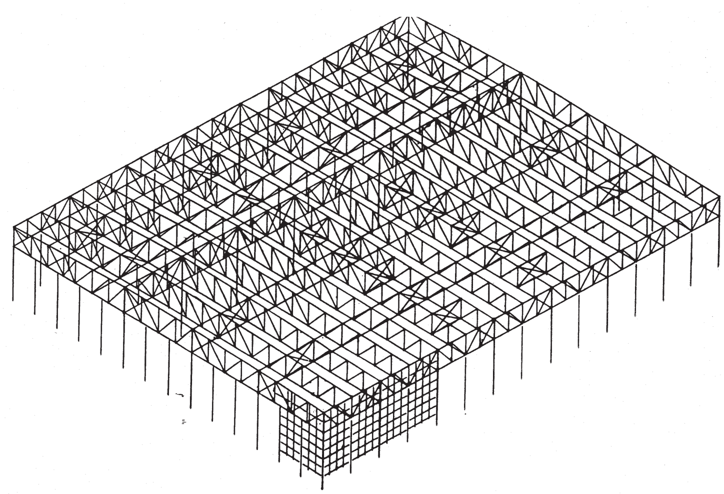

Fig. 4-13. Case Study One framing—original design.

which long-span joists can span easily. The sway frames, spanning 50 ft between roof trusses, were selected with the knowledge that they could be installed in pairs and redesigned to support the adjacent 20-ton underhung craneways. The roof joists were panelized, including roof deck, and installed in 1,000 ft^2 sections. A summary of the bid results for the original and alternative framing concepts, shown in Figure 4-15, exemplifies the savings gained.

The alternative framing system was 28% lighter and 15% stiffer than the original framing scheme. Jack trusses were reduced from 17 needed to only two, while the roof trusses went from 44 needed to only 14. In addition, the number of field bolts was reduced by 15,000. The only element that increased in number was the bottom chord bracing members, which allows for a better distribution of lateral forces throughout the structure.

The alternative framing and installation scheme delayed the project start by four weeks to allow time for the redesign. However, the redesign simplified exterior foundations and grade beams, reduced the cost of the structural steel framing by $400,000, and reduced the overall construction schedule by five weeks, allowing the project to be completed within the owner's original schedule.

This is an excellent example of how constructability was used to improve the project and the building. The slate was clean. A 100,000 ft^2 manufacturing facility was required and the design/build team, with several years of design and construction knowledge and expertise, generated a better solution for the owner.

4.8 CASE STUDY TWO

A high-bay 120,000 ft^2 manufacturing facility was designed with a roof system consisting of roof trusses at 50-ft spacing with continuous sway frames and bottom chord bracing around the perimeter bays and across the building at the third points. Due to the height of the facility, accompanying wind and seismic forces, as well as operational loading from the underhung cranes, the lateral forces at the bottom chord level were significant.

This building had been designed, the construction contracts were awarded, and the steel fabricator was in the process of material procurement when the general contractor went out of business. The steel fabricator contacted the owner and proposed a team approach to revise and save the project. This idea was implemented, and a solutions-oriented atmosphere was created.

The discussions centered on the owner's criteria, design drawings, and work completed to date, including that the majority of the steel had been ordered and would be in the fabricator's yard within four weeks. The time lost due to the default of an original project participant was also a concern because the facility start date was critical. Furthermore, even a small delay of the project would create a conflict for the fabricator's shop time, which had been scheduled to accommodate other projects.

Through a collaborative effort by the entire team, constructability became the central topic of discussion. The bracing scheme was identified as an area of significant labor time and potential savings to preserve the project completion date.

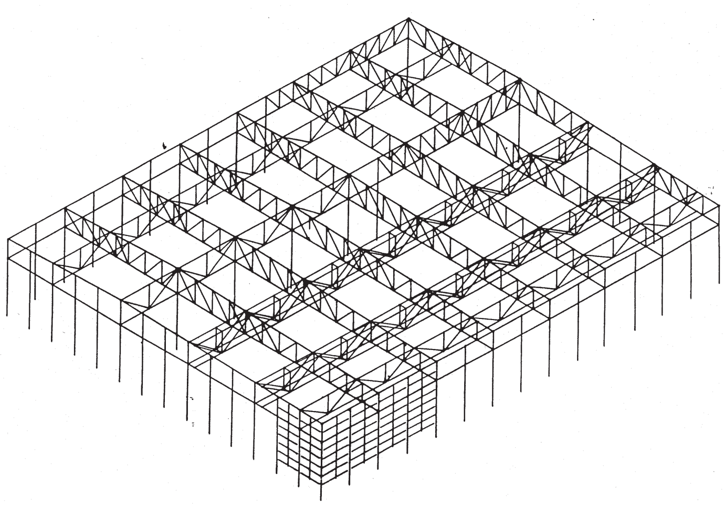

Fig. 4-14. Case Study One framing—simplified alternate design.

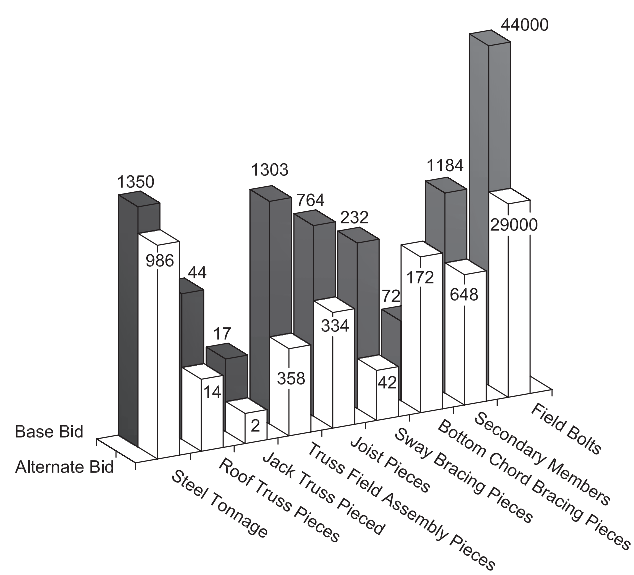

Fig. 4-15. Comparison of costs for Case Study One.

It was discussed that the bracing requirements and connection details had become very costly and time consuming in the field. In the original bracing scheme, shown in Figure 4-16, the design of the X-brace connections and the preparation of the shop details would require the skills of a seasoned engineer and detailer. The shop fit-up required was not a significant issue, except for the number of pieces involved. The common bolts through the chord web created an erection concern that might have become a safety issue in the field. The connection required more bolts because the detail material could not be attached to the chords in the shop, requiring bolts in the chords and bolts for the diagonal bracing as well as bolts for the splice at midspan (Figure 4-17). The length of the bracing members was also a concern. The erection sequence was troubling because the bracing members had to be erected after the purlins in order to install the rod hanger. And finally, the plumbing of the building would have been a concern once the bracing connection was made.

The team conceived an alternative diamond bracing system (Figure 4-18), where the diamond braces were located from mid-point to mid-point of the bay to be braced. Field labor was significantly reduced and shop labor was also streamlined. In addition, the truss bottom chords that are subject to high axial forces due to lateral forces were provided with an intermediate brace. An increase in the number of pieces improved erection because the pieces were shorter and easier to stabilize and install, since they were independent pieces. In addition, the number of field bolts was reduced and the rods to the purlins were eliminated.

Design and detailing for the diamond bracing scheme was also simplified, as were fabrication and installation, since the brace can be installed on top of the gusset plate and the field bolted connections can be made without interference with other members. See Figure 4-19.

Although it didn't begin this way, by necessity this project found a basis in constructability, which was the essential element that saved the project in a crisis.

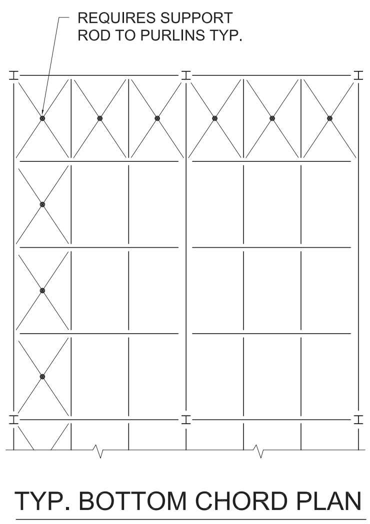

Fig. 4-16. Case Study Two framing—original design.

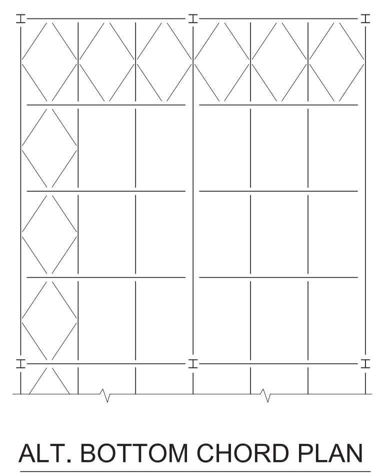

Fig. 4-18. Case Study Two framing—simplified alternate design.

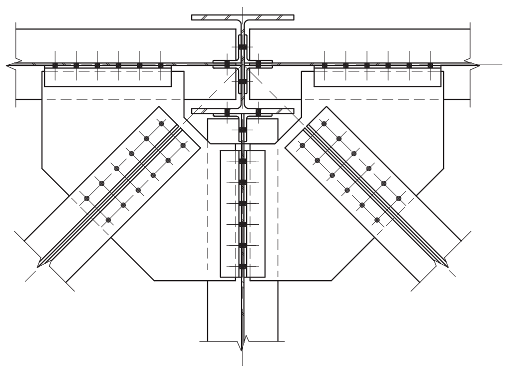

Fig. 4-17. Case Study Two—original X-bracing detail.

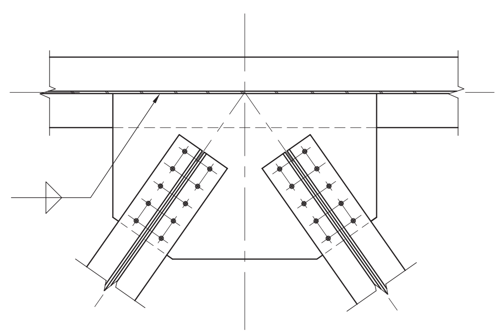

Fig. 4-19. Case Study Two—simplified alternate diamond brace detail.

Chapter 5
Detailing and Fabrication

When implemented, constructability can enhance the project through more accurate and cost-effective proposals based on complete and coordinated design documents. This, in turn, can improve the entire structural steel detailing and fabrication process through economical material procurement, timely shop drawing preparation, and conformance to standard shop fabrication processes and OSHA requirements. Constructability can also improve connections when basic detail considerations are understood and accounted for early in the process. Following is a discussion of each of these areas.

5.1 COMPLETE AND COORDINATED DESIGN DOCUMENTS

Design drawings that are complete, dimensioned accurately, and coordinated with the architectural drawings will reduce the time required to produce responsive bids. Showing complete connection designs for lateral-load-resisting systems in the construction documents will allow more competitive bids, as each fabricator will be pricing the same connection configurations. However, when the connection designs are not shown, the fabricator must determine and price the minimum connection that he or she believes will meet the structural engineer of record's design intent. This may expose the owner to apparent low bids that only later flood the project with requests for information (RFIs), change orders, and extras. Rarely, during the bidding process, is there sufficient time for fabricators and detailers to investigate and fully develop connection information for accurate pricing.

Ambiguous statements such as "connect for full moment capacity," "provide stiffeners and doublers as required," "column splices shall be connected for the full bending capacity of the smaller shaft," and "column splices shall develop 125% of the tension capacity of the member" only increase the risk borne by the fabricator, which in turn increases the prices from responsive bidders without adding value to the project. This inflation of the structural steel costs often puts the project itself at risk and results in so-called value engineering that cuts away at the meat of the project to reduce costs, leaving bad feelings among all parties. The owner is disappointed, the SER is insulted, the construction manager is aggravated, and the successful contractor is seen as the opponent, when, actually, the devil was in the (omission of) details.

When complete connection designs are indicated on the design drawings, the fabricator may be given the option to submit alternative connections that meet the original design criteria, subject to the requirements in Section 4.2 of the *Code of Standard Practice*. When acceptable, such alternative connections often enable the fabricator to alter the connection configurations to meet the shop practices (standards) without compromising the integrity of the structure.

5.2 MATERIAL PROCUREMENT AND SHOP DRAWING PREPARATION

The advanced bill of materials, also known as the mill order, is traditionally prepared by the fabricator's order office or shop detailer. Note that the mill order is usually placed as shop drawing preparation begins as a means to nest activities and expedite the fabrication process. This also helps to ensure that material will be available in the shop upon approval of the shop drawings. Thus, design drawings that are complete and coordinated also allow the preparation of a more accurate and complete mill order. The ability to order material quickly and accurately enables the fabricator to enhance production and delivery schedules.

This procurement process can be further streamlined, and often its accuracy enhanced, when the project is designed and virtually constructed in a building information modeling (BIM) approach. This 3-D design model can be translated into a shop detailing computer model, allowing the fabricator to quickly and accurately prepare an advance bill of materials. This enhancement has been shown to reduce the overall cost of the material and provide the owner with more competitive pricing.

With the advent of computer designs, members are increasingly sized to be the least-weight shape required to support the loading conditions. This often results in many different beam and girder sizes throughout the floor plan. Yet, the intended economy may not be realized if this selection approach results in small quantities of each shape used.

Constructability discussions can foster a standardization of the shapes throughout the floor plan for similar framing conditions into fewer different sizes with quantities that exceed the mill-order minimums. This practice will also save time and cost in detailing, fabrication, erection and inspection.

When a project is "fast-tracked," changes in the structural design can occur at any time during the design process. To minimize the exposure of the owner to the costs of changes in the fast-track process (see Section 3.6 in the *Code of Standard Practice*), fast-tracking should be discussed in the constructability review and all parties must be aware of the schedule for completion of design phases.

The importance of coordinated and complete design drawings to the process of preparing shop drawings cannot be overstated. A BIM approach can also improve the shop drawing process. A 3-D model meeting the format of CIS/2 can save a significant amount of time in the production of shop drawings. CIS/2 is a standard data format enabling 3-D modeling design/analysis software to pass required information to 3-D detailing software for use in preparing shop drawings. The model can be imported into the steel detailing software saving weeks in detailing time. However, the model must be complete and coordinated to achieve these benefits.

Sometimes only minor changes occur and there is a temptation not to alter the 3-D analysis model, as long as the construction documents are changed. But, even minor changes are critical for proper detailing. Thus, when a BIM approach is used, the model should always be updated, even for minor changes.

Constructability discussions should include how the RFI process will be managed. The RFI process has been used by detailers and others to document and record the inquiries and associated responses related to interpretation and implementation of the construction documents during the normal course of preparing shop drawings, and by fabricators and erectors to resolve field issues, communicate errors in construction, and seek approval for corrective measures. Section 4.6 of the *Code of Standard Practice* provides guidance for the proper use of the RFI process. All parties to a contract will benefit from a successful implementation of the RFI process.

While the number of RFIs on a job does not necessarily indicate the quality of the design documents, it can be stated that incomplete and uncoordinated construction documents usually result in a large number of RFIs that can involve far more than simple clarification. This RFI process can be time consuming and labor intensive for all parties involved and should not be abused. Just as the documents should be coordinated and complete prior to steel detailing, RFIs should be written only to ask legitimate questions or concerns, and RFIs should be resolved as soon as possible.

An accurate 3-D model makes all framing conditions easy to visualize and can also save the detailer and the SER time in the approval process. In transmitting the 3-D model and having it imported directly into the detailing software, the SER can be assured that each member and end reaction in the steel detailing model match the design model. This should facilitate the approval process.

The approval process may be shortened even more by minimizing or eliminating the transmission of drawings between the SER and the steel detailer with on-line approval of electronic shop drawings. The approver can use a detailing-software-compatible "viewer" to review the detailing model and the related shop drawings, apply mark-ups to the members, and e-mail the approval comments to the steel detailer for corrections. This process can reduce the approval time to just a few days for each submittal.

5.3 STANDARDS

Standards exist for almost all aspects of design and construction. For steel design and construction, the basic documents include:

- *Specification for Structural Steel Buildings* (ANSI/AISC 360) (AISC, 2005a), referred to as the AISC *Specification* in this design guide
- *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) (AISC, 2005b), referred to as the AISC *Seismic Provisions* in this design guide
- *Code of Standard Practice for Steel Buildings and Bridges* (AISC 303) (AISC, 2005c), referred to as the *Code of Standard Practice* in this design guide

The AISC *Specification* references other major standards, including the RCSC *Specification*, the *Structural Welding Code—Steel* (AWS D1.1/D1.1M) (AWS, 2008), hereafter referred to as AWS D1.1, several ASTM material specifications, and other documents applicable to steel design and construction.

The AISC *Steel Construction Manual* (AISC, 2005d) compiles the requirements into a handbook format with design aids, design guidance and standard recommendations for details and systems. These recommendations enable an efficient approach for a typical project. Alternative details can be used, but discussions in the constructability review should be used to determine the best approach for the project.

The typical connection details shown in the AISC *Steel Construction Manual* are known and usually allow the detailing process to proceed more efficiently. Custom connections may require an added in-depth assessment by the detailer and fabricator when developing the job standards prior to preparing the shop drawings. Typical connections that meet the shop standards of the fabricator allow the efficiency of use of fittings and components that have been stockpiled by the fabricator. Typical connections will also reduce SER approval time since the connections will match the industry standards. If there are standard connections that are not acceptable to the SER, these should be clearly noted in the construction documents and discussed in the constructability review.

5.4 CONNECTIONS

Connections, whether bolted or welded, can have a significant influence on constructability. Constructability considerations can be instrumental in determining the connection types and fastening methods that will be of greatest benefit to the project.

In-depth information on connections in general is available in the AISC *Specification* and the AISC *Steel Construction Manual*. Further information on bolting can be found in the RCSC *Specification;* AISC Design Guide No. 17, *High-Strength Bolts—A Primer for Structural Engineers* (Kulak, 2002); and the RCSC *Guide to Design Criteria for Bolted and Riveted Joints* (Kulak, Fisher and Struik, 2001). Further information on welding can be found in AWS D1.1 (AWS, 2008); *Structural Welding Code—Seismic Supplement,* AWS D1.8 (AWS, 2005), hereafter referred to as AWS D1.8; and AISC Design Guide No. 21, *Welded Connections—A Primer for Engineers* (Miller, 2006), hereafter referred to as Design Guide 21.

5.4.1 Bolted Joint Considerations

Some common considerations to discuss include:

- What are the preferred connections and configurations for the job?

- Do the joint details allow for the access necessary to insert and install the bolts?

- Are twist-off bolts advantageous?

- What corrosion protection requirements apply, and what fastener options make the most sense? In galvanized construction, only ASTM A325 bolts are an option because ASTM A490 bolts cannot be galvanized. If weathering steel such as ASTM A588 is used for the structure, ASTM A325 or A490 Type 3 weathering steel fasteners should be used. Galvanized fasteners cannot be used because the small amount of zinc will attempt to protect the rest of the exposed steel and be quickly consumed.

- What bolted joint types should be used? Note that the bolt installation and inspection requirements follow once the joint designation is established by the SER. The joint types and appropriate uses are summarized in the RCSC *Specification* Section 4. Snug-tightened joints will be less expensive than pretensioned bearing joints. Slip-critical joints are the most expensive, and usually significantly so.

- What are the paint requirements? What faying surface preparation is required?

- Do the details and framing configurations require the use of oversized or slotted holes?

- Can the threads be excluded (X condition) in the joint designs to reduce the number of bolts?

- Can the number of different types of bolts used on a job be reduced to one or two? If two, do the details use different diameters to simplify the process of ensuring that the stringer bolts are placed in the right holes?

- Are the bolt diameters and corresponding pretensions required practical for the job? The force required to pretension ASTM A490 bolts larger than about 1-in. diameter, or to pretension ASTM A325 bolts larger than about 1¼-in. diameter, exceeds the capacity of most commonly used pneumatic impact wrenches.

- What pretensioning method is to be used when pretensioned joints are required? The available methods are provided in RCSC *Specification* Section 8.2, with corresponding pre-installation verification requirements. As all methods can be used successfully if used properly, the contractor should be asked to select the most economical and efficient method based upon the sizes, types of connections, material costs, available equipment, logistics, and experience of the bolting crew.

- Are the bolt lengths long enough for the common pre-installation verification devices, or are special procedures required? Some bolt pretension indicating devices cannot be used to verify the performance of short fastener assemblies. Bolts too short for a bolt tension indicating device may be tested in any convenient steel plate with the use of direct tension indicators (DTIs) provided the production lot of DTIs is first verified using a longer bolt in the bolt tension indicating device.

- What inspection requirements are applicable? Inspection should be in accordance with Section 9 of the RCSC *Specification*, and Chapter 17 of the *International Building Code* (ICC, 2006), hereafter referred to as the IBC, provides specific requirements for the "special inspection" of high-strength bolts and their installation. Note that torque measurement is not a reliable method for inspecting bolts.

5.4.2 Welded Joint Considerations

Some common considerations to discuss include:

- What are the preferred connections and configurations for the job?

Table 5-1 Economy of Weld Types for Various Joint Types		
Joint Type	**Most Economical Solution**	**More Expensive Alternatives**
Lap	Fillet weld	Plug or slot weld
Butt	PJP groove weld	CJP groove weld
Tee and corner	Fillet or PJP groove weld*	CJP groove weld
* As the fillet weld size gets larger, a PJP groove weld may be more economical.		

- What joint types should be used? Table 5-1 indicates the comparative economies of type of weld, assuming typical conditions, for various joint types and loading cases. Selection of the particular groove weld detail should be left to the steel contractor, based upon the selected welding process and the position of the weld. Depending upon base metal thickness and throat required, the steel contractor should select the root opening and groove angle combination that gives the best combination of cost, ease of welding, and resultant quality. As an example, in thicker materials, narrow groove angles are more difficult to weld and thus require a higher skill level, but require less filler material, require less time, and reduce weld shrinkage and angular distortion. Distortion control is another reason welding design may be best left to the steel contractor.

- Can the joint designs and details be oriented for horizontal and flat position welding? The position of welding (flat, horizontal, vertical or overhead), and in the case of tubular joints, whether rotated or not, has a significant effect upon weld cost.

- Do the joint details allow for the access necessary to make the welds? If, for example, a pair of parallel stiffeners are closely spaced, inadequate access may not allow the proper angle for the welding gun or electrode, leading to weld quality and weld size issues. Typically, a welding gun or electrode angle of least 30° must be provided to make a quality weld. Maybe a single stiffener can be used instead of the pair of stiffeners.

- What welding processes and filler metals are appropriate? The fabricator and erector are in the best position to select the appropriate welding process for a given project, location and weld, based upon the amount of welding to be performed, the equipment available, the skills of welders available, the position and access for the welding to be performed, and the quality and performance requirements for the weld or welds. AWS D1.1 (AWS, 2008) permits several welding processes, including:

  (1) Four prequalified processes:
      (a) SMAW (Shielded Metal Arc Welding).
      (b) FCAW (Flux Cored Arc Welding).
      (c) GMAW (Gas Metal Arc Welding).
      (d) SAW (Submerged Arc Welding).

  (2) Four code-approved processes that require qualification testing:
      (a) ESW (Electroslag Welding).
      (b) EGW (Electrogas Welding).
      (c) GTAW (Gas Tungsten Arc Welding).
      (d) GMAW-S (short-circuit transfer mode of GMAW).

- What quality and inspection requirements are appropriate?

- What welding procedure specifications (WPS) are most advantageous? WPS are prepared by the steel contractor to provide the welders the information necessary to make a quality weld. Most steel contractors work with prequalified WPS, which require no physical testing for weld quality or mechanical properties. When staying within the prescribed limits of prequalification provided in Section 3 of AWS D1.1, and following the manufacturer's written recommendations for use, often called operating characteristics, the steel contractor writes the WPS. If the steel contractor elects to exceed AWS D1.1 prequalification limits, or to work outside the bounds of the manufacturer's recommendations, a WPS can be qualified by testing.

- What Charpy V-notch (CVN) toughness requirements are applicable? Generally, these requirements are dictated by the AISC *Specification* and AWS D1.1, or the AISC *Seismic Provisions* and AWS D1.8.

Table 5-2. SSPC Surface Preparation Levels				
Surface Preparation Levels*		Typical Construction	High-Performance Coatings	Specialty Coatings
SSPC–SP 1**	Solvent Cleaning	X	X	X
SSPC–SP 2	Hand Tool Cleaning	X	—	—
SSPC–SP 3	Power Tool Cleaning	X	—	—
SSPC–SP 5	White Metal Blast Cleaning	—	—	X
SSPC–SP 6	Commercial Metal Blast Cleaning	—	X	X
SSPC–SP 7	Brush-Off Blast Cleaning	—	X	—
SSPC–SP 10	Near-White Blast Cleaning	—	X	X

* Other levels are defined by SSPC but are not common in shop fabrication.
** Solvent cleaning is a part of all other cleaning methods according to SSPC, though it may bear repeating in a project specification.

- Do joints indicated with a weld-all-around symbol work? It is common for the "weld-all-around" symbol to be used by SER's on design drawings. This practice is uniformly discouraged by the steel construction industry. Welding the entire perimeter of the connection is rarely required and may lead to problems when welders must change planes (change position) when turning corners. This frequently causes undercut of the corner edges, poor weld profile, and undersized welds, with a higher risk of crack initiation at these corners. It is better to specify the minimum weld size, length, and location of welds required between the connected parts.

- What fillet weld termination requirements apply? Requirements and options are provided in AISC *Specification* Section J2.2b.

- What inspection requirements are applicable? Inspection should be in accordance with AWS D1.1 (AWS D1.8 for high-seismic applications) and Chapter 17 of the IBC provides specific requirements for the "special inspection" of welding.

- What nondestructive testing requirements are appropriate? Extensive guidance on proper selection of the non-destructive testing (NDT) method is provided in Design Guide No. 21. General welding inspection should focus on inspection activities before and during the welding operations, rather than following completion of welding.

5.5 SURFACE PREPARATION

Constructability discussions also should explore surface preparation and coating requirements so that all parties to the contract understand what is—and is not—required.

Foremost, it is important to understand that surface preparation and coating is not required for most structural steel. Steel that is to be covered up by interior finishes does not require any surface preparation or coatings to be applied. This steel will not deteriorate during the normal life of a building. Steel that is to receive spray-applied fire-resistive material (SFRM) should not receive surface preparation other than removal of dirt, grease, loose mill scale, or other material that would impair adherence of the SFRM. The profile of tight rust developed by exposing the steel to the normal atmosphere during storage and erection will help the SFRM to adhere more securely.

Steel located within an exterior wall that may be subjected to moisture during the life of the building does require protection, however. Over a long period, unprotected steel that is moist most of the time will corrode and may experience significant section loss. Long term protection may be necessary; although the best answer, if practicable, is to design, detail, construct, and maintain the exterior facade to eliminate the moisture condition.

Surface preparations for structural steel are specified by The Society for Protective Coatings (SSPC). It is important when specifying surface preparation for high performance coatings to refer to the SSPC visual guides, such as SSPC-VIS-1, to determine what to expect to see when the surface preparation method is performed on the structural steel. Each preparation method has its appropriate uses and cost implications. Uses for each surface preparation are summarized in Table 5-2. The first four methods are common in typical construction.

Surface preparation that develops the appropriate profile for paint to properly perform is usually specified by the paint manufacturer. The most commonly specified surface preparations are:

- SSPC–SP2 Hand Tool Cleaning
- SSPC–SP3 Power Tool Cleaning
- SSPC–SP6 Commercial Blast Cleaning

Steel that is only required to be shop primed can be adequately cleaned in the shop by SSPC–SP2 or SSPC–SP3 without any further surface preparation. These are the least expensive of the surface preparation methods. SSPC–SP6, Commercial Metal Blast Cleaning, is the next most expensive and commonly specified surface preparation. Note that there are additional costs with blast cleaning, over and above the paint material and application labor, associated with the process, including inspection, drying time in the shop, and extra care in handling and shipping. The constructability review should be used to discuss project requirements, coating and preparation options, and which choices most benefit the owner in terms of performance, budget, and schedule.

Other surface preparations that are less commonly specified, generally listed in order of increasing cost, include:

- SSPC–SP1 Solvent Cleaning
- SSPC–SP7 Brush-Off Blast Cleaning
- SSPC–SP10 Near-White Blast Cleaning
- SSPC–SP5 White Metal Blast Cleaning

Conventional shop primers are of minimal benefit, even for short-term protection—they are simply a bonding layer that should be used only when a topcoat subsequently will be applied. There are various levels of quality and durability of shop primers, depending on the types(s) of additional coats and the conditions under which the steel will be exposed in the final application. The compatibility of the shop coat and subsequent coatings should be discussed in the constructability review.

Also, when multi-coat systems are selected, the constructability review should explore whether to apply multiple coats entirely in the shop or apply one or more in the field. If all painting is done in the shop, the responsibility for normal field touch-up to repair damage occurring during shipping and erection should be addressed in the contract documents. As required in the contract documents, the fabricator will exercise special caution in handling, loading and shipping to minimize such damage, but touch-up should be expected and a suitable contractor assigned responsibility for same. If there is concern that the touch-up painting will not blend well with the final coat, it is best to have the final coat applied in the field by a coating subcontractor after erection is complete.

Rather than discuss the many choices in manufacturers and paint types, the reader is referred to the SSPC literature, literature available from coating manufacturers, and coating/corrosion consultants who are excellent sources for what coatings to specify for various applications.

5.6 GALVANIZING

Galvanizing can also be used in lieu of painting, although galvanizing is usually similar in cost to higher-end paint systems. During the galvanizing process, the steel is immersed in a zinc bath, which bonds the molten zinc to the surface of the steel.

The primary method for cleaning surfaces at the galvanizing facility is to immerse the steel in a chemical pretreatment bath. When chemical cleaning is not anticipated to be effective due to the presence of mill lacquer, paints, markings, or weld slag, abrasive blasting may be suggested. A minimum of a SSPC–SP7 Brush-Off Blast Cleaning surface preparation is required. The blasting process increases the preparation costs and time for galvanizing. Galvanizing plants have varying kettle sizes and in general it is most economical to design members and assemblies that can be galvanized in a one step, "single-dip" process. "Double-dipping" is required if the length of the piece exceeds the kettle size.

Extra caution must be taken when galvanizing cold-worked steel. Many structures and parts are fabricated using cold-working techniques (bending, hole-punching, rolling, shearing). This is commonly seen in bent anchor rods and handrail joints. Another potential problem area is the bends in square and rectangular HSS shapes. Severe cold-working increases the incidence of strain-age embrittlement, the effects of which may be accelerated by the galvanizing process. Strain-aging is relatively slow at ambient temperatures but more rapid at the elevated temperatures encountered in the galvanizing process. The visible result of strain-aging embrittlement is cracks which form in the bend area after galvanizing.

Design and detailing for effective galvanizing must allow for proper drainage for cleaning solutions and for the flow of molten zinc into, over, through and out of the fabricated member without undue resistance. Vent holes are also critical to allow for the release of gases to prevent potential explosions from occurring as heat expands the trapped gas. Failure to provide for this free, unimpeded flow can result in complications for the galvanizer and the customer. Improper drainage design results in poor appearance, bare spots, and excessive build-up of zinc. All of these may require repair and are unnecessary and costly. These drainage or vent holes are also required in HSS. If an HSS is used for a column, the bottom of the column and cap plate must have holes placed in them for drainage and release of gases. Usually the cap plate is covered up by concrete or beams so it is not necessary to fill the holes.

Drainage and vent holes are required when galvanizing handrail. Holes must be placed in inconspicuous places so they are not readily visible in the finished product.

In all of these cases, the holes should remain in the finished piece, if at all possible. The plugging of drainage or vent holes is costly. Since the zinc will flow within the item, the galvanizing will be present not only on the outside surface of the member but also the inside surface.

Galvanized steel can be easily and effectively coated with compatible paints, not only for aesthetic purposes, but also to extend the structure's service life. The age and extent of weathering of the galvanized coating dictate the extent of surface preparation required to produce a quality paint system over galvanized steel. ASTM D6386, *Standard Practice for Preparation of Zinc (Hot-Dip Galvanized) Coated Iron and Steel Product and Hardware Surfaces for Painting* (ASTM, 2005), should be consulted for suggested surface preparation methods for galvanized coatings of varying ages.

5.7 SHIPPING AND DELIVERY

A proper constructability review will guide the design to be compatible with shipping and delivery considerations.

There are limitations to the size of shipping pieces. In general, trucks are 8 ft wide and normal trailers are about 40 ft long. Loads in excess of these limits can be shipped, but with increasing difficulty. Overlength and overwidth loads will usually require permits and may require one or two escorts. They may also be restricted in some jurisdictions as to the times they can move. Shipping solutions for loads as wide as 14 ft or as long as 100 ft have involved the use of specialized equipment such as steerable pups and extendable trailers.

On most roads and highways, the maximum total weight that a standard trailer may carry is 40,000 lb. Typically, this can only be exceeded if the trailer is equipped with additional axles. In northern climates during the spring thaw, the axle loads permitted on the roads may be reduced to prevent damage to the roads (frost laws). The frost laws restrict the allowable trailer weight, which in turn increases the number of required loads and ultimately the associated costs.

Many times it is thought that structural steel can be erected right off the truck. In reality, this is a costly option, requiring special blocking and inefficient loads. Normally, structural steel is loaded, interlocked and strapped into place for safe transport over the road. This procedure ensures that the load does not become dislodged during transit. When the steel arrives on the job site, it must be unloaded and shook out and arranged in an orderly fashion per the prescribed sequence of installation.

Chapter 6
Constructability and Steel Erection

This chapter presents aspects of the on-site construction process that affect cost and schedule and that can be improved with consideration of constructability. This chapter outlines the erection process and offers suggestions that allow for informed decision making, reduced costs, improved schedules, and less conflict between the design and construction teams.

6.1 COMPLETE AND COORDINATED DESIGN DOCUMENTS

Design drawings that are complete, dimensioned accurately, and coordinated with the architectural drawings will reduce the time required to produce responsive bids. The specifications and the drawings should clearly differentiate between the separate segments of the industry. This allows the individual contactors to clearly define the scope of work upon which to quote, and ultimately to perform.

All of the structural steel should be shown and specified on the structural drawings. This includes anchor rods, embeds, or anchors, etc., required to erect the structural frame. Metal deck and stud shear connectors should be shown and specified on the structural drawings. Members and frames that participate in the lateral load resisting system should be identified as required in Section 7.10 of the *Code of Standard Practice*.

The architectural drawings should show and specify all of the miscellaneous metals and the ornamental iron. Sheet metal studs may be shown on the structural or architectural drawings but not specified on both. The concept preferred by the steel contractor is "first pass" erection: the construction of the facility by a succession of trade contractors without backups, return visits, or interdependence of trades. This concept views the facility in succession from design through construction as foundations, structure, stairs, floor slabs, fascia, and mechanical/electrical/plumbing (MEP) systems, thus allowing for minimal interferences by other trades. This is an ideal constructability model.

6.2 SITE CONSTRAINTS

The typical job site is shared by many diverse contractors, including steel, concrete, miscellaneous metals, ornamental iron, metal deck, shear connectors (studs), curtain wall, and others. The coordination among trades, including deliveries and installation schedules, can only be accomplished when the construction documents have been properly prepared; coordinated among the disciplines; and released in complete, trade-specific bid packages.

If all structures could be built in an open field on sound, solid, well-drained, engineered fill with only one contractor working on the site at a time, there would be no issues. That is, of course, never the case, and job site considerations must include the adequate access, work space and storage space as outlined in Section 7.2 of the *Code of Standard Practice*.

The allowable bearing capacity of the surface soil must be strong enough to support the cranes, trucks, man-lifts and other equipment, as well as providing a satisfactory storage/laydown area for marshalling the steel for erection. If this is not the case, cribbing and engineered fill may need to be brought in to the site. The site conditions must allow the steel contractor to establish a safe and proper work area in order to meet OSHA jobsite safety requirements.

- What is the available work space and storage space on site?
- What are the soil conditions on site? Can they be improved if necessary?
- What construction sequence, if any, is preferred or required for the project?
- Would it be better to field-assemble components (such as a large truss)?

6.3 SEASON OF CONSTRUCTION

Considering the location of a project and the weather that is likely (and unlikely) to be experienced during steel erection is extremely important. Construction issues for a building erected during hurricane season in Florida will be totally different than those for the identical building erected during winter in Minnesota. The actual considerations for each project will vary; however, there are some general questions that should be addressed before the start of any project, as follows:

- Can construction be scheduled so that a "weather window" of opportunity for steel erection minimizes the chance for interruption?
- What is the risk that bad weather could create unsafe conditions?

- In what season will the field painting be done?
- Should the top coat of the coating system be applied in the shop or field?
- Is the project field-welded, and is there sufficient budget to provide temporary heating in the field, if necessary?

Construction is a continuous operation performed during all seasons and in every climate with tremendous variations in temperature, precipitation and wind velocities across the country. Seasonal issues can affect constructability and may be beneficial to consider when developing the concept and final design. These same issues should again be reviewed by the construction team when developing the on-site project plan.

6.4 SEQUENCE OF CONSTRUCTION

The planning and scheduling of steel erection operations are extremely important in achieving constructability. Only by proper planning can the sequence of construction be developed that will provide the most efficient, safest manner in which the structure can be built, consistent with the original design assumptions and the appropriate quality standards.

In structural steel erection, like manufacturing, the most efficient process is "just in time" delivery of material. The objective is to minimize the handling of the structural steel, allowing sections to be fabricated, delivered to the site, and erected quickly and safely in sequence. Constructability ensures that just-in-time delivery will work.

Ideally, during erection, the crane will progress from one end of the building to the other, setting each piece of steel in sequence with no comeback work. After the material is set by the raising gang, the plumbing and detail gangs bolt, plumb, weld, etc., the structure in the same sequence. The metal deck and shear studs are installed closely behind the detail gang, followed by the placing of the concrete slabs. The other trades follow, with everyone working the project plan for sequencing on schedule and within budget.

The detailer or the fabricator's engineering department develops accurate mill order lists and prepares accurate shop, erection and fieldwork drawings that are consistent with the sequence of delivery agreed upon by the general contractor, fabricator and erector. The mill material is ordered in sequence and in sufficient advance time to ensure the material will arrive at the shop prior to the release of the approved shop drawings for fabrication. If the steel is being procured directly from the mill, this activity must be coordinated with the mill's rolling schedule. Constructability review makes this process flow efficiently.

In theory, the detailer prepares the shop drawings per sequence, checks the drawings, and forwards them to the fabricator and then to others for approval. Upon approval, each sequence that is complete and, without "holds," is issued to the shop for fabrication. When there are "holds" on areas, information required, or changes noted on the drawings, it will not be possible for the fabricator to complete the sequence as planned. The process becomes inefficient for the detailer, fabricator and erector, as well as the SER. It is also more prone to errors and likely to cause delays on the project. The automated detailing and fabrication software programs of today, and the advent of building information modeling (BIM), have revolutionized the industry and raised the bar of quality and accuracy; but they do not solve the problems that occur with design changes made after drawings are released for construction—or with incomplete construction documents.

Since the fabricator has ordered the material in sequence and may have ordered the beams in multiple lengths from the mill, starting the fabrication of a sequence without all of the approved shop drawings requires double handling of the material, revised scheduling, increased shop inventory, and perhaps even delay in tracking down the material in the shop.

The fabricator must complete and ship each sequence consistent with the original plan for the proper installation of the structural steel. The erector receives, shakes out and installs the structure by sequence. Any pieces that are missing or out-of-sequence may require extra field work, cause lost time (for the erector and all following trades), and lead to extra cost and schedule delays.

If the general contractor wishes to control the direction or sequence of structural steel installation, the general contractor must include such information within the construction documents as specified in Section 7.1 of the *Code of Standard Practice*.

6.5 CONSTRUCTION SCHEDULE

The construction team's planning and scheduling are directly benefited by constructability input. Planning is the key to any successful project, and scheduling is a major component of the planning process. Every task, for each trade, has a preceding task that must be properly executed. If these tasks can be logically assembled and executed, it is less likely that the project will experience delays and/or added costs. These concepts become extremely important when structural systems are intermixed, such as structural steel and precast columns and steel girders, precast shear walls, precast plank, and masonry infill. This concept may be economical on paper; however, the economy can only be realized if constructability is the driver of the planning, coordination and scheduling throughout the design and construction processes.

6.6 OSHA REQUIREMENTS

When the design and construction team are well-informed, and the project is properly planned and good construction procedures are being executed, the OSHA 1926 Subpart R (OSHA, 2008) requirements for steel erection safety are likely to be implemented. Most of the provisions in this governmental regulation of construction activities concern the work of the general contractor and the steel erector. A few also impose requirements on the SER and others. Some of these are summarized below.

The steel detailer may be asked to provide bolt holes to attach temporary bracing or OSHA-required perimeter safety lines. The fabricator may be asked to install special lifting devices or erection aids, which may require additional holes in the permanent steel, subject to the review and approval by the SER.

Repairs, alterations and modifications to anchor rod placement must be approved by the SER, and it is the general contractor's responsibility to inform the steel erector that this task has been performed. Some of the pertinent OSHA provisions follow:

- Column bases must have a minimum of four anchor rods. The SER and detailer must be aware of this when designing and detailing the base plate to foundation connection.

- Bolts common to two opposing connections cannot be used in the webs of columns, unless temporary support is provided for the first beam erected. A minimum of one bolt must maintain the connection of the first member. Many details are available to eliminate the sharing of bolts. Alternatively, erection seats or extra holes in one of the connections could be used.

- The site conditions provided by the general contractor must be firm, level and drained to support the erection equipment. The total area under and around the structure must be adequate to support the man-lifts.

- The steel erector must maintain stability of the structural frame at all times during steel erection.

OSHA also requires that the controlling contractor must certify that the concrete has reached adequate strength to begin erecting the structural steel.

These requirements are all good common sense items that must be considered in developing the project plan and must be executed by the construction team.

6.7 SPECIAL ERECTION PROCEDURES

Should the SER's design concept require unique or special erection procedures, the SER must define the concept and its requirements within the contract bid documents. Examples include when the SER's design concept requires structural steel installation in a specific sequence or when shores or jacks must be adjusted as erection progresses. Figure 6-1 shows an example of sequenced jacking of a suspended roof system.

The concept definition, the assumed erection sequence, the dead load deformation information, and the acceptable fabrication and installation tolerances must be defined by the SER within the construction documents. Structures such as long cantilever spans for amphitheaters, flexible roof systems, unusual or complex structures, and moveable structures and their supports fall into this category. An example of special lift hitches used to raise a heavy assembly from horizontal to vertical and for subsequent installation is shown in Figure 6-2.

Fig. 6-1. Sequenced jacking of a suspended roof system at Ford Field, Detroit, MI (photo courtesy of SmithGroup).

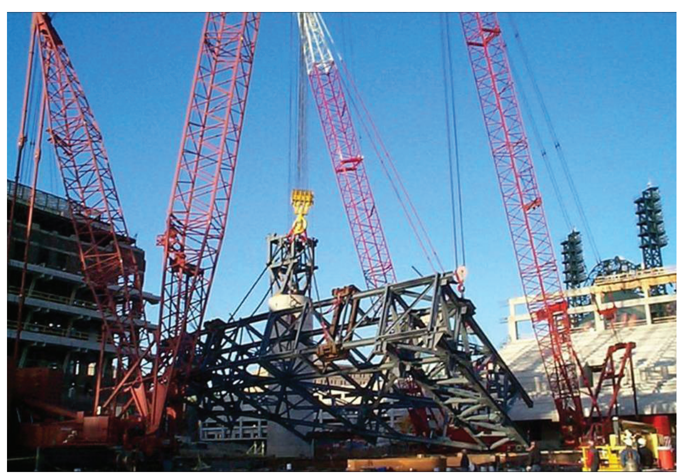

Fig. 6-2. Heavy assembly lifted from horizontal to vertical at Ford Field, Detroit, MI (photo by Ruby + Associates, Inc.).

Special attention must be given when the structure will support heavy equipment loads during erection or a special erection procedure is used. The erector's engineer must analyze the structure for these conditions and provide temporary support where necessary. In some cases, it may be that members must be increased in size or additional framing added to sustain the erection-induced loads, at the erector's expense. The erector's engineer must review and analyze the structure, develop an erection sequence and procedure in compliance with the SER's original design assumptions, and coordinate with the SER during the review of the proposed erection procedure. Sharing of computer models and open discussions of major assumptions, connection design criteria, temporary bracing, and stability concerns are necessary ingredients to the success of the project. Consideration should be given by the SER to allow temporary members within the structure, used for the special erection procedure, to remain in the finished structure, perhaps reducing the overall project cost and schedule. In such cases, direct communication between the SER and SSE can be very helpful.

6.8 TEMPERATURE ADJUSTMENTS

Temperature variations result in expansion and contraction of steel. Since structures are subject to wide variations in temperatures resulting from both the air temperatures and direct exposure to the sun, the SER should establish a base temperature for the structural design. This will allow the construction team to detail, fabricate and erect the structure consistent with the base temperature. This may be a non-issue for most buildings, but for long spans or large trusses, or when intermediate or final elevations or installation tolerances are critical, temperature plays a big role in the development of the installation procedure and in the procedure that is used to confirm the results. Imagine a 500-ft truss system (which grows almost 0.4 in. for each 10 °F) weighing 2,500 tons being lifted vertically to frame between two fixed supports and the reader will understand the importance of establishing a base temperature.

Structural steel will begin to absorb the heat from the sun at sunrise and continue through sunset. The exposed shapes will grow and lead to a phenomenon called "sun cambering." The building continues this cycle each day, varying based on ambient temperature, cloud cover, construction progress, deck and slab installation, and exterior cladding—which often prompts the erector to plumb the structure when the steel temperature has normalized.

6.9 SPECIAL TOLERANCES

When special fabrication and installation tolerances are required to meet special design criteria, they must be clearly indicated within the construction documents by the SER. Such tolerances will require the fabricator and erector to develop the means and methods, including necessary shop or field adjustments, to allow the structural steel to be installed within these special tolerances. Upon award, the steel detailer will prepare the shop drawings accordingly and will indicate the tolerances and adjustment information on the erection drawings. Note that mill rolling tolerances are not negotiable; therefore, special fabrication and installation tolerances should be developed within the boundaries of ASTM A6 (ASTM, 2007).

The tolerances for all the construction materials must be considered in the design, planning and execution of the Contract Documents. Steel, concrete (including precast), fascia, masonry and foundations are just a few that must be identified and included in the constructability decision matrix. When considering the standard tolerances for the various materials and their interface with other materials, special tolerances must often be developed in order to provide for the constructability of the final structure. These special tolerances do not come for free. Optional framing schemes should be reviewed; however, when the options have been exhausted, the owner should be made aware of the reason for and the potential added costs due to these special tolerances.

6.10 ERECTION STABILITY

The *Code of Standard Practice* and OSHA 1926 Sub Part R (OSHA, 2008) both state that the erector is responsible for stability of the structural steel during erection. During the bidding stage, stability may be the most difficult issue for the steel erector to recognize and subsequently develop sufficient information to establish a probable cost.

The *Code of Standard Practice* states in Section 7.10.2 that the owners designated representative for construction (ODRC) shall indicate to the erector, prior to bidding, the installation schedule for nonstructural steel elements of the lateral load resisting system and connecting diaphragm elements identified by the SER in the construction documents. From Section 7.10.1, the SER must also identify any special erection conditions or other considerations that are required by the design concept, such as the use of shores, jacks or loads that must be adjusted as erection progresses to set or maintain camber, position within specified tolerances, or prestress.

Section 7.10.3 outlines the erector's responsibilities. The erector must determine, furnish and install temporary supports or other elements required for a stable erection operation. These temporary supports must be sufficient to secure the bare structural steel framing against loads that are likely to be encountered during erection, including those due to wind and those that result from erection operations. The erector need not consider loads during erection that result from the performance of work by, or the acts of, others, except as specifically identified by the SER and ODRC, nor those that are unpredictable, such as loads due to tornado, earthquake, explosion or collision.

The stability of members and frames during handling and erection at the construction site must be investigated by the erector. OSHA states that "structural stability shall be maintained at all times during steel erection." The initial stability of columns has been enhanced by the OSHA requirement of a minimum of four anchor rods connecting the column base to the foundation.

Most girders, as designed, are stable only when their compression flange is laterally supported. The possibility of lateral buckling of girders during installation must be investigated by the erector. An approximate determination of the stability of the girder may be made by dividing its length by its flange width. As a rule of thumb, most girders with l/b less than 80 will be stable during erection; for values greater than 80, the erector should consider some form of temporary support during and/or after the lift. Note that this ratio is not a substitute for an engineering analysis. The erector may make similar checks for truss top chords and other long and limber assemblies during handling and installation.

The steel fabricator should note that this criterion for stability also applies to handling of girders and trusses in the shop. The SER may consider using it to establish a preliminary flange width for girders or truss top chords. With the option of sizing the truss chords for lateral stability, the cost of the additional material can be compared against the cost of the special handling and temporary lateral bracing necessary for the smaller chord.

A series of trusses designed to be braced by purlins may be installed in a condition where braced points do not exist. The first truss in the series generally will have no means of bracing for the top chord when it is released from the crane. Bracing must be installed so that the truss chord is stable until the next truss, sway frame, purlins, and possibly even the top and bottom chord bracing are installed.

Stability issues exist to a varying degree in all structures. It may not be economical to design members so they can be installed without temporary bracing. However, the options can be discussed during the consideration of constructability concepts in order to develop the best balance between design, fabrication and erection economy.

There are many ways to rig building components so that they remain in a stable and undamaged condition during installation. Generally speaking, economy in rigging and installation stability occurs when the fewest measures have to be taken to ensure stability. In short, installation economy is attained by:

- Minimizing temporary shoring.
- Minimizing temporary guy wires.
- Minimizing temporary guy anchors.
- Minimizing rigging and lift fixtures.
- Simplifying the rigging.
- Reducing amount and size of construction equipment.

Each of these issues is influenced by the initial design decisions made by the owner and design team. Yet they are not responsible for means and methods of construction. Based on the owner's and architect's requirements, and the SER's resulting design, the structure may be easy or difficult to fabricate and/or erect. The concepts of constructability can be used to promote design decisions that are made through proper construction knowledge and input.

AISC Design Guide No. 10, *Erection Bracing of Low-Rise Structural Steel Frames* (Fisher and West, 1997), details how to determine the need for erection bracing of various framing members and also provides some prescriptive systems for temporary bracing. *Code of Standard Practice* Section 7.10 and Commentary discusses the requirements for the SER regarding identification of elements of the lateral load resisting system as well as special erection conditions required by the design concept.

Identify early those items in a project that may create potential material handling, global or local stability issues, or the reliance on multiple trades for final stability. Following is a list that could aid in the evaluation by the owner and design team of a structural design for material handling and stability issues.

- Evaluate construction sequencing at the 25% review phase of design.
- Identify potential areas where handling and stability may create a constructability issue.
- Prepare preliminary schematic installation scheme for a portion of the structure.
- Prepare a temporary stability plan for this area that provides a temporary lateral load resisting system during construction.
- Evaluate critical members for their partially supported dead load capacities.
- Perform an initial cost estimate for the temporary stability requirements.
- Determine in what parts of the structure the requirements for temporary stability measures could be reduced to provide cost saving options to improve the constructability.

6.11 GENERAL ERECTION TOLERANCES

Code of Standard Practice Section 7.13 describes in detail the erection tolerances that have been developed through long-standing practice. These tolerances were first established in the 1924 edition of the *Code of Standard Practice* and were revised in 1959 to reflect many years of experience and changes in construction practice. The incentives for obtaining acceptable tolerances as steel erection progresses are to achieve a safe and serviceable structure and to maintain the construction schedule. These are two of the most critical elements of project constructability.

It should be noted that these tolerances are workmanship standards, and were they to be exceeded, the structure might still perform satisfactorily provided structural stability and building serviceability requirements are met. Where smaller tolerances are required for architecturally exposed structural steel (AESS), building finish, or serviceability requirements, these tolerances should be clearly listed in the project specifications. Special tolerance requirements may result in an increase in fabrication and erection costs, and it is advisable to seek input from fabricators and erectors with experience on similar projects to find the most economical way to achieve the tolerances needed.

The erection tolerances in the *Code of Standard Practice* are defined relative to member work points and working lines. In order to survey a structure to determine its position, it is necessary to correctly and accurately locate these work points and working lines on the structure. Because they are sometimes located either at the centroid of the section or at the intersection of two planes, such as the center of the web at the top of a flange, special care must be used to locate offset points that can be surveyed. In addition, steel structures during the erection stage are quite flexible and subject to movements due to wind, dead loads and temperature.

Dead load effects on column splice locations are typically only a concern in high-rise structures. Differential column shortening may be a consideration for columns in lateral load resisting frames and wind mullion type columns, which typically have substantially less axial stress. The SER should compensate for differential column shortening by directing the fabricator to lengthen the column detailed length as required to achieve equal length for all columns under load.

A special case of differential column shortening occurs in buildings with reinforced-concrete shear cores. For buildings up to about 30 stories, the shrinkage and creep for the shear core will be similar to the column shortening for fully loaded gravity columns, and adjustment is often not required for differential shortening. The problem arises with certain types of concrete core construction systems where the elevation for each lift or floor is set by distance from grade and not the floor immediately below. This results in substantial compensation for concrete shrinkage, which is the majority of the shortening effect for the concrete. When this method of elevation control is used for the core, the steel columns should be detailed long to compensate for their shortening under load so that floor elevations remain level.

The *Code of Standard Practice* tolerance for elevation of floor steel is determined by a measurement from the column splice, not by a theoretical elevation from grade. In high-rise structures, the column splice locations may be surveyed for both location and elevation at every tier. Where significant variations in elevation are noted, the column elevations may be adjusted either by shimming or adjusting the length of columns in the upper tiers. Defining "significant variation" in elevation is difficult and will depend on span and location in the structure. When *Code of Standard Practice* tolerances have been exceeded, differences up to ½ in. between adjacent columns and 1 in. across the breadth of the structure have been accommodated without serious serviceability problems. It may be difficult to accurately survey the tops of columns in an open high-rise structure, and this is especially true where more than one instrument setup is required.

Thermal effects, especially differential thermal effects, may be very difficult to compensate for when surveying a structure. Surveys of structures (especially high-rise structures) should be performed when the structure is at a uniform temperature. The temperature of the steel will normalize between 3:00 a.m. and 5:00 a.m., limiting the time available for plumbing and establishing control lines. As soon as the sun begins to warm the structure, the structure will move away from the exposed surface, and it will continue to move throughout the day and into the evening until it normalizes again at about 3:00 a.m. the following day.

Thermal movement is a function of temperature differential, building stiffness, and the height of the structure. Each structure will present its own set of issues to be addressed in order to perform the plumbing operation and/or establish accurate elevation control.

In order to apply temperature correction to survey measurements, it is necessary to know not only the ambient temperature of the steel but also the location of the center of rigidity of the structure. This is the neutral point for thermal expansion and is usually located at a brace point, such as at steel bracing or the shear core. For a moment frame, it might be necessary to know the varying stiffness and location of the frames. The center of rigidity may vary significantly as the steel structure is erected and fascia, concrete and other materials are installed. The erector and field surveyor may need input or analytical assistance from the SER to locate the center of rigidity in complex structures.

Code of Standard Practice Section 7.13 discusses the design, detailing and installation of items designated as "adjustable items" on the construction documents. Typically, these are edge forms, curtain wall attachments, and elevator and escalator supports. The SER is responsible for determining the amount of adjustment needed to achieve the specified tolerance. The fabricator then must detail and fabricate a connection that will provide this adjustment, and the erector must install it to the required tolerance. The tolerance specified in the *Code of Standard Practice* is ±⅜ in. vertical from the column splice line and ±⅜ in. from the established building finish line at the particular floor.

Disputes can arise because of the difficulties in establishing the building finish line. Steel frames without concrete floors in place or exterior walls installed often will move more than ⅜ in. due to the effects discussed above. These disputes can be avoided if grid lines are established at each level by the owner or the owner's designated representative for construction before the adjustable items are installed. All trades should then use these same grid lines to lay out their work.

General contractors are often reluctant to lay out grid lines before placing the concrete on the floors because this requires offset reference lines on the structural steel in order to preserve the lines. This additional effort is well spent, however, because this is the only way to make sure all trades are working to the same building finish line. To promote this, the owner's designated representative for design may require survey control in the construction documents to establish one common set of building finish lines.

Chapter 7
Special Constructability Issues

This chapter includes many special topics that are relevant to constructability.

7.1 ANCHORAGE TO CONCRETE

Connections are an extremely important part of the structure, and a vital contributor to constructability on the project. Most field problems—and failures—occur due to inadequately designed, fabricated or erected connections. This tendency is exacerbated because the process of anchoring steel framing to other materials introduces other trades, tolerances and quality standards into the successful completion of the structure. Connections between steel and other materials should be completely designed by the SER and clearly located and detailed in the construction documents. In addition, any erection sequence requirements and installation tolerances must also be defined within the construction documents in order for the general contractor to properly sequence the construction. Anchorages require careful attention by the SER, fabricator, erector, and the trades responsible for their installation.

Connections of structural steel to concrete are required in many applications including:

- Roof truss to tilt-up concrete wall.
- Column bases to foundations.
- Embed plates for shear and tension.
- Renovation of existing facilities.
- Reinforcing of existing beams and columns.
- Modifications of existing foundations.

There are two general types of anchors: cast-in-place and post-installed. Cast-in-place anchors are the anchor of choice for the majority of new construction. Types of cast-in-place anchors include anchor rods with hex heads or double nuts, hooked rods, J-bolts, and stud-welded plates (see Figure 7-1). Post-installed anchors are placed after the concrete has set. They can be used for many purposes, including as a repair for mislocated or missing anchors, in the retrofit of existing construction, or as accurately set anchors for new construction. Types of post-installed anchors include shell anchors, wedge anchors, undercut anchors, adhesive anchors and grouted anchors (see Figure 7-2).

7.1.1 Anchor Rods

For erection safety, four anchor rods are required for all column base plates as specified by the OSHA *Safety and Health Regulations for Construction,* 29 CFR 1926 Subpart R, Section 1926.755(a)(1) (OSHA, 2008). They are required as a safety anchor during installation primarily to resist forces generated at the top of the column due to wind or accidental impact, prior to connecting the column to horizontal steel framing. The standard states:

> Each column anchor rod (anchor bolt) assembly, including the column-to-base plate weld and the column foundation, shall be designed to resist a minimum eccentric gravity load of 300 pounds (136.2 kg) located 18 in. (0.46 m) from the extreme outer face of the column in each direction at the top of the column shaft.

It is possible, however, that during erection, a horizontal force could be applied at the top of a column that would cause a much higher load on the anchor rod assembly. Although it may be difficult to foresee and quantify this load, it

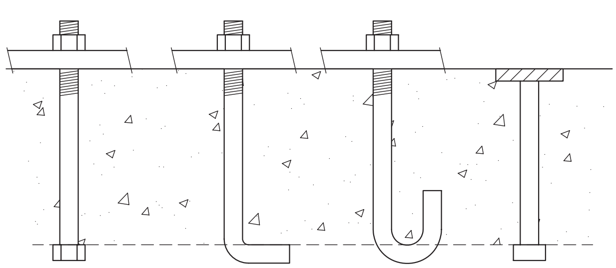

Fig. 7-1. Cast-in-place anchors.

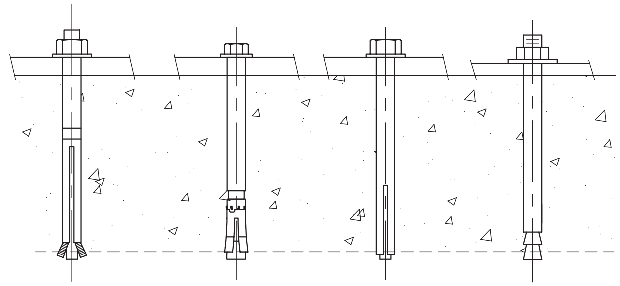

Fig. 7-2. Post-installed anchors.

should be considered by the erector during the development of the installation procedure.

This anchorage requirement for column bases does not apply to a "post." A post is defined by the OSHA standard as "a structural member which has a longitudinal axis that is necessarily vertical, and which either weighs 300 pounds or less and is axially loaded, but is laterally restrained by the above member. Posts typically support stair landings, wall framing, mezzanines and other substructures."

AISC Design Guide No. 1, *Column Base Plates* (Fisher and Kloiber, 2006) contains an excellent summary of materials, fabrication, installation and repair of anchor rods, as well as design. Some of the issues dealing with repair, alteration or replacement of anchor rods are discussed below.

Section 7.5 in the AISC *Code of Standard Practice* addresses installation and tolerances governing anchor rods, foundation bolts, and other embedded items. Because these installations require more than one trade, they are prone to additional problems. Field fixes are generally accomplished with the use of post-installed anchors or modification of the original misplaced anchor rod. However, proper design, detailing, and preparation of the concrete can greatly reduce the field errors.

It is important to note that the OSHA standard for steel erection safety requires that the SER review and approve any repair, modification, or replacement of anchor rods. In addition, prior to setting columns, the steel erector must receive from the general contractor written confirmation that any repairs or modifications have been made and approved.

One of the primary causes of anchor rod mislocation is the lack of coordination within the original construction documents. Typically the structural drawings are separated into several bid packages with minimal coordination between the sections. The concrete drawings will include the column pier design with rebar sizing and placement requirements. The base plate and anchor rod designs are located on the column schedule within the steel portion of the project design drawings. It is likely that the specified rebar and anchor rods cannot be placed as specified without interfering with one another. Thus it is recommended that the SER complete the detail, indicate the location of the rebar and anchor rods, and consider the installation tolerances for each material to ensure that the design and construction packages are coordinated and constructable. An example of coordinated anchor rods and column pier reinforcement is shown in Figure 7-3.

Anchor rods are sometimes set with their tops lower than the detailed elevation. The anchor rods can be placed so low that the top of the anchor rod is below the top of the base plate and a nut cannot be engaged, or the anchor rod may extend above the base plate but not enough to permit full thread engagement. A sleeve coupling can be used with a short threaded rod extension, or it may be necessary to cut off the anchor rod and weld a threaded extension in place to provide the proper projection as shown in Figure 7-4.

In locations requiring an anchor rod extension with very limited space, a detail similar to Figure 7-5 can be used. To design this detail, assume the rod weld is a partial-joint-penetration groove weld and size the pipe section and weld to complete the joint. Anchor rod material should be selected as a weldable material to allow for ease of field modification, in case other means or methods for modification are not possible. This detail can also be used as a repair when the anchor rods have been eaten away by corrosion. To help avoid the situation of anchor rods with little or no projection above the top of the base plate, the SER can specify and the steel fabricator can detail the anchor rods with additional projection (beyond the AISC *Code of Standard Practice* tolerance, 3 in. or more).

Foundation and anchor rod inaccuracies have also been addressed by Ricker (1989a). He suggests that out-of-tolerance alignment will exist and that most column base plates should be furnished with oversized holes, allowing small displacements of anchor rods to be tolerated. These recommendations are the basis of the current anchor rod hole sizes in the AISC *Steel Construction Manual* (AISC, 2005d).

Anchor rods that are tilted (not vertical) can sometimes be straightened with a rod bending device. For anchor rods displaced up to about ¾ in. (or 15° with vertical as noted in Figure 7-6), the concrete may be chipped away to a depth of

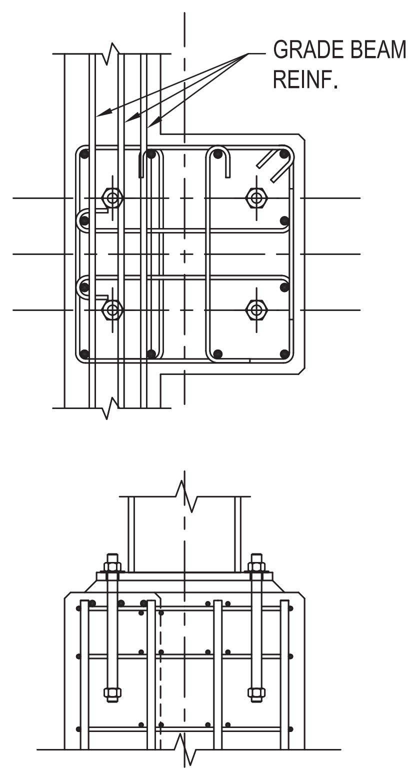

Fig. 7-3. Coordinated anchor rod and rebar locations.

a few inches, the anchor rod bent into proper position, and the foundation regrouted (see Figure 7-6).

Heating the anchor rods prior to bending may not be an acceptable practice. The axial capacity of high-strength steel anchor rods may be compromised by the heating process. The SER must be consulted if such practice is deemed necessary.

Anchor rods displaced over ¾ in. usually require that the base plate be slotted. Severe error may locate the anchor rod outside or near the edge of the base plate. Edge distance is usually not a problem as long as the base detail is adequate to transfer the design forces. Heavy plate washers (½ in. or greater) with offset holes are used to cover the slots. These are welded to the top of the base plate in the field (see Figure 7-7).

Anchor rods may also be displaced toward the interior of the base plate. There are several methods to correct this situation, but all are costly. The errant anchor rod can be cut off at the surface of the concrete and epoxy anchors or expansion bolts set into cored holes at their proper locations, if room permits. Such drilling, however, is often complicated by the presence of the reinforcing steel, and the SER must determine if it is permissible to sacrifice the reinforcing should one or more rods be encountered.

If an entire anchor rod grouping is misplaced but the individual anchor rods are in their proper relationship, it may be possible to offset the column base plate from its original location.

Anchor rods that are displaced too near the edge of the column flange may require the column flange to be notched

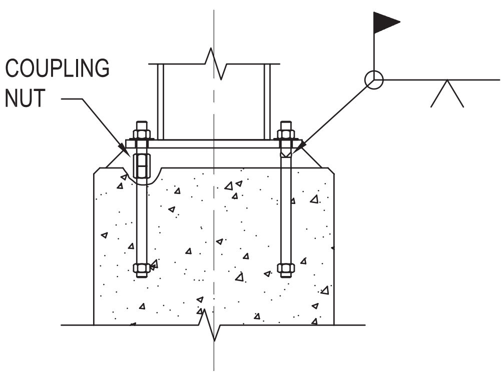

Fig. 7-4. Typical anchor rod extension.

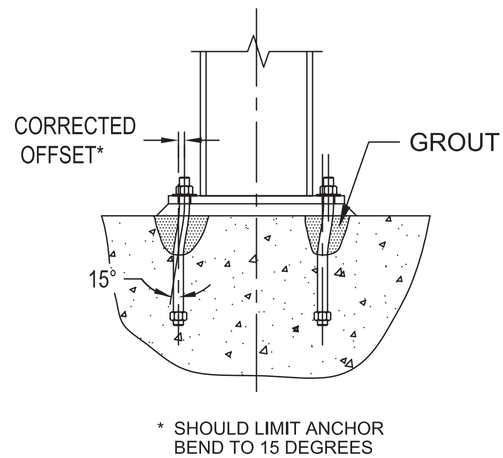

Fig. 7-6. Anchor rod correction—slight mislocation.

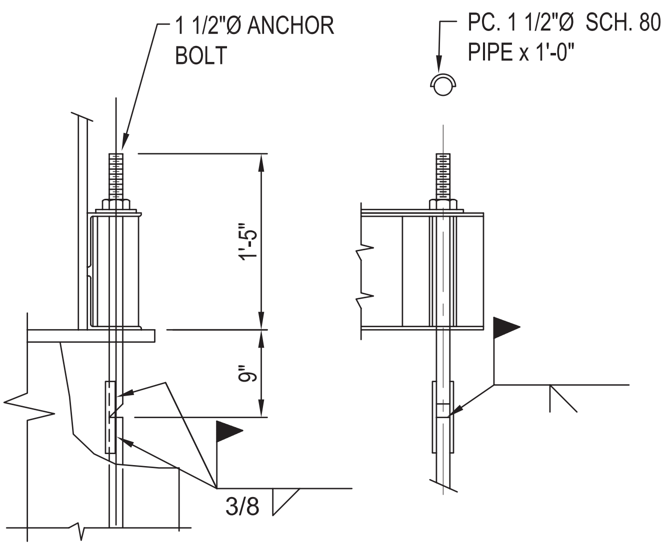

Fig. 7-5. Anchor rod extension—limited space.

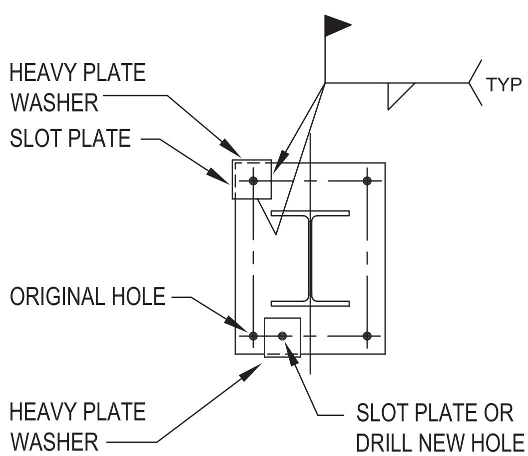

Fig. 7-7. Anchor rod correction—severe mislocation.

to allow the anchor rod nut to tighten. This slight loss of bearing area may not be a problem, but it should be investigated and approved by the SER prior to notching the column.

If possible, the SER should specify only uniform, square anchor rod patterns. This will prevent the anchor rods from being placed within a pattern that is rotated 90° from the detailed orientation.

Anchor rods subjected to a corrosive environment may be galvanized. These should preferably be ASTM F1554 Grade 36 or 55 material to avoid embrittlement issues with higher strength steels. When ordering galvanized anchor rods, the fabricator/erector should specify that the threads must be "chased" to allow the nuts to be installed. Weathering steel may be used in normal atmospheric environments, although the rods will rust during weathering and stain the foundation concrete.

7.1.2 Anchor Rod Tolerances

The interface between two materials that have different tolerance requirements presents special problems. The most common tolerance problem in building construction is anchor rod placement. ACI 117 Section 2.3 (ACI, 2006) covers placement of embedded items and allows a tolerance on vertical, lateral and level alignment of ±1 in. This tolerance is significantly larger than that specified in the AISC *Code of Standard Practice* Section 7.5.1, which lists the following tolerances:

- The variation in dimension between the centers of any two anchor rods within an anchor-rod group shall be equal to or less than ⅛ in.
- The variation in dimension between the centers of adjacent anchor-rod groups shall be equal to or less than ¼ in.
- The variation in elevation of the tops of anchor rods shall be equal to or less than plus or minus ½ in.
- The accumulated variation in dimension between centers of anchor-rod groups along the established column line through multiple anchor-rod groups shall be equal to or less than ¼ in. per 100 ft, but not to exceed a total of 1 in.
- The variation in dimension from the center of any anchor-rod group to the established column line through that group shall be equal to or less than ¼ in.

While the AISC *Code of Standard Practice* anchor rod tolerances are readily achievable, reports from the field indicate they are often exceeded. The AISC *Code of Standard Practice* tolerances for anchor rod placement should be specifically written into the project specifications rather than referenced, in order that the foundation contractor is made aware of these requirements.

Since OSHA requires any anchor rod modification to be reviewed and approved by the SER, AISC in Design Guide No. 1, *Column Base Plates* (Fisher and Kloiber, 2006) and in the AISC *Steel Construction Manual* have recommended relatively large base plate holes to provide additional clearance. These holes and the recommended washers are shown in the AISC *Steel Construction Manual* Table 14-2.

Proper installation of anchor rods provides for a safe, fast and economical erection of the structural steel frame. The installation process starts with the preparation of an anchor rod layout drawing. The structural steel detailer should coordinate all anchor-rod details with the column-base-plate assembly. This layout drawing will show all anchor rod marks along with layout dimensions and elevation requirements. Because of schedule pressures, there is sometimes a rush to set anchor rods using a drawing submitted for approval. This should be avoided and only approved released-for-construction placement drawings should be used for this important work.

Recommendations for laying out anchor rods are:

- Where possible, use a symmetrical pattern in both directions.
- Use as few different layouts as possible.
- Typical layouts should be four anchor rods (OSHA Subpart R mandated) in a square pattern.
- Anchor rod layouts should provide ample clearance for the washer from the column shaft and its weld, as well as a reasonable edge distance. While there is no specified minimum edge distance requirement for anchor rod holes, it is recommended to provide a minimum of approximately ½ in. of material from the edge of the hole to the edge of the plate.
- Where more strength is required, consider using larger rods with diameters up to about 2 in. or more, rather than high strength rods.
- Keep the construction sequence in mind when laying out anchor rods adjacent to walls and other obstructions. Make sure the erector will have the access necessary to set the column and tighten the anchor rods. Where special settings are required at exterior walls, moment bases and other locations, clearly flag these settings on both the column schedule and foundation drawings.

- Anchor rod details should always specify ample thread length. A minimum of 3 in.—and preferably more—greater than the projection above the base plate should be specified to allow for variations in setting elevation.

Fast-track projects and projects with very complex layouts present special problems if the steel design and detailing lag behind the initial foundation work. On these projects, it may be better to use drilled-in epoxy type anchor rods rather than cast-in-place anchor rods. However, the SER must confirm that the anchor rods can be installed with minimal interference with the reinforcing steel.

Templates should be made and used by the construction team for all of the various settings. Typically, templates are made of plywood on site. The advantage of plywood templates is they are inexpensive to make and are easy to fasten in place to the wood foundation forms. The anchor rods can be held securely in place and relatively straight by using a nut on each side of the template. Steel templates consisting of flat plates or angle type frames are sometimes used for very large anchor rod assemblies requiring close setting tolerances. Steel plate templates can also be reused as setting plates.

Embedded templates are sometimes used with large anchor rod assemblies to help maintain alignment of the rods during concrete placement. The template should be kept as small as possible to avoid interference with the reinforcing steel and concrete placement. With the embedded template, the anchor rod assembly has to be placed first and the reinforcing steel placed around or through the template. Care must be taken to carefully vibrate the concrete around the template to avoid hollow spaces or voids in the concrete. This is especially important if the template serves as part of the anchorage.

Layout of all anchor rods should be done by an experienced construction surveyor. The surveyor should be able to read structural and layout drawings and understand industry construction practices. A licensed land surveyor may or may not have the necessary knowledge and experience.

In summary, the concrete contractor, steel detailer and fabricator, general contractor, erector, and the SER must understand anchor rod tolerances and the importance of accurate placement. It is essential for the safe and timely erection of a steel frame that will meet all AISC and project requirements.

7.1.3 Embeds

It is recommended that embed anchors be designed for a minimum of 2 in. of eccentricity in each direction (Figure 7-8) and that the embed plate be made up to 6 in. larger in each dimension to allow for relocation due to rebar interferences. The SER should review the rebar and stud requirements and design the embed plates accordingly.

Where the fabricator has been assigned the task of designing the embed plates, the eccentricity requirement should be included in the SER's connection design requirements. However, based on material tolerances, possible field problems, and constructability issues, it is recommended that the SER completely design these embed plates and include the necessary details within the contract documents. This recommendation is especially pertinent if the embed connections are governed by seismic design provisions. It is also recommended that the procedure required by the OSHA standard (OSHA, 2008) for the SER's approval of repair, modification or replacement of anchor rods be applied to embeds as well.

7.1.4 Embed Tolerances

Steel beam connections are made to concrete using bearing plates or embedded plates. The actual in-place tolerances of cast-in-place concrete, especially concrete core shear walls, vary from ACI tolerances, depending on the forming system used and the ability to accurately brace the forms. Experience has shown that basement walls and tall columns, as well, often exceed the listed ACI (2006) tolerances. Modern jump formed walls may be able to achieve a tolerance less than the ACI requirements.

If conditions allow use of a seated connection, this is usually the most straightforward way to provide the field adjustment needed. If the beam is not located on the top of the concrete member, however, a seated connection may not be an option because forming contractors are reluctant to provide notches or voids in their form. The more common type of connection is an embedded plate flush with the wall and with a field welded connection to the beam.

Embedded plate connections should be sized to allow a substantial horizontal and vertical adjustment of the steel connection. Using a large plate, sized to accommodate a minimum of a 2 in. variation, will allow concrete anchor spacing that will develop the full strength of the anchor and

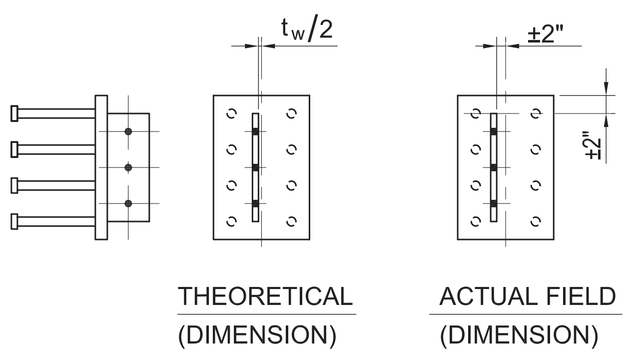

Fig. 7-8. Embed plate eccentricities.

reduce the effect of any eccentricity due to a reasonable error in placement.

When the concrete connection is an embedded plate, the beam side of the connection has to allow for the full ACI concrete tolerance of 1 in., or as near to this as possible. The maximum adjustment that can be supplied using a double-angle connection field welded to the embedded plate and field bolted with long slots in the beam (see Figure 7-9) is approximately ±⅝ in. This may be enough adjustment if the concrete contractor is careful in placing the forms and if the steel frame plumbing tolerances can accommodate some of the embed tolerance. If additional adjustment is required, it may be possible to use a similar connection on the opposite end of the beam. When more design strength is required or additional length adjustment is needed, the connection can be changed by field welding the connection angles to the beam instead of bolting.

When connections to concrete require skewed connections, they can be made using a skewed single plate, a single or double bent plate connection, or special seated connection. Double bent plates, while difficult to fabricate, are still the preferred connection when length adjustments are required and the loads exceed the capacity of a single skewed or bent plate. The double bent plates can be designed and detailed similar to the double angle connections with slots that were discussed above.

For purposes of constructability (primarily safety and schedule issues), proper anchorage and connection of steel framing to concrete, masonry, or other materials must be given a high priority by the entire design and construction team. The previously discussed list of potential problems is foreseeable, and the team should have a plan for dealing with misalignment, dislocation, or out-of-tolerance installation.

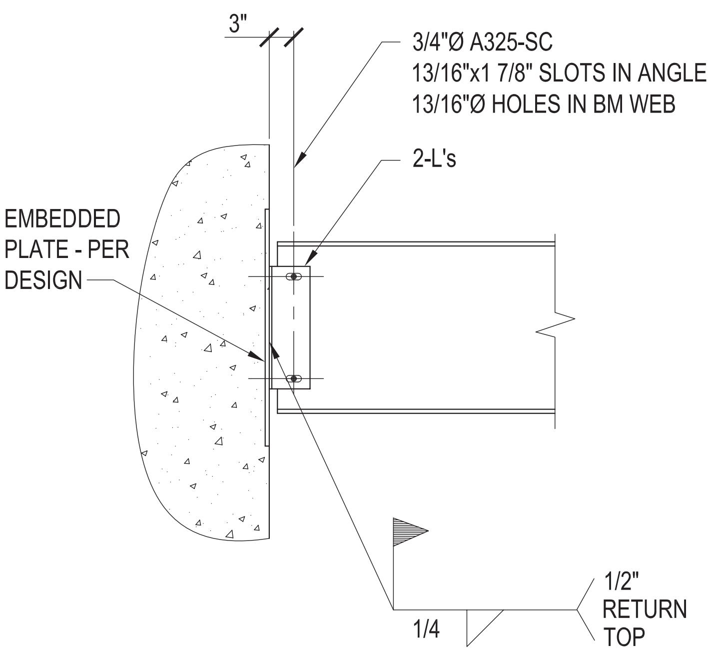

Fig. 7-9. Shear connection attached to embed plate.

7.2 CAMBER

The *Code of Standard Practice* Section 6.4 specifies camber tolerances for rolled shapes. For members less than 50 ft long, the camber tolerance is minus zero/plus ½ in.; an additional ⅛ in. per each additional 10 ft of length (or fraction thereof) is allowed for lengths in excess of 50 ft. AWS D1.1 (AWS, 2008) specifies similar tolerances for welded built-up members.

The cambering of a beam or girder is primarily a means to counteract some portion of the deflection. Depending upon the case, when camber is used it may be appropriate to camber for partial dead-load deflection, full dead-load deflection, or full dead-load deflection plus partial live-load deflection. The considerations when specifying camber go further than this, however.

As with any product, the permitted variations must be considered. Additionally, there are considerations that add to the potential for deviations. By necessity, camber is measured and confirmed in the shop in the unstressed condition (see *Code of Standard Practice* Section 6.4.4). Yet handling, shipping and erection operations can affect the camber that remains when the beam is in-place in the structure.

Ricker (1989b) provides an excellent summary of cambering and the associated considerations, as well as guidance for the specification of camber. This paper provides an excellent basis upon which the use of camber can be discussed. Having such a discussion in the design phase before camber is specified is a key way to foster mutual understanding of what is needed and what can be done, as well as achieving a successful outcome.

Some additional recommendations when considering camber include (Ricker, 1989b):

- Members to consider cambering include filler beams, interior girders, composite floor beams, members with uniform cross section, trusses, and crane girders greater than 75 ft.

- Members *not* to camber include crane girders less than 75 ft, spandrel beams, beams with cantilevers, beams braced with knee braces, members of nonuniform cross section, beams with unsymmetrical loading, beams subject to torsional loads, beams that will be moment connected, beams less than 25 ft in length, and beams less than 14 in. in depth.

- Specify camber of beams and girders only if ¾ in. or more. When the beams are very light, the camber should be restricted to a minimum of 1 in., due to the likelihood that the beam will lose camber during shipping, receiving and installation.

- Camber is likely to be lost in any fabricated product during the handling, shipping, receiving and installation process due to stress or strain relaxation, dead load deformation, connection restraint, or other field imposed loads. See Section 6.4 of the *Code of Standard Practice*.

- Don't over-camber beams that will receive shear studs for composite action. Depending on the method of concrete placement, over-cambering may result in the heads of the studs protruding from the top of the concrete slab.

- Cambering of members with web thicknesses less than ¼ in. can result in web buckling.

- Don't camber beams to which a cover plate will subsequently be welded. The heat thus generated at one flange will generally be enough to significantly alter the camber curve.

- Columns comprised of section sizes normally associated with beams should be noted as "columns" with "no camber permitted" when ordered from the mill. Otherwise, the mill may provide the members with a natural mill camber.

- Heat cambering should be performed only on low-carbon steels. Application of heat to medium- and high-carbon steels increases the danger of embrittlement. ASTM A36, A572 Grade 50, and A992 are examples of low-carbon steels.

For additional discussion on camber, see Ricker (1989b).

7.3 FABRICATION TOLERANCES

Section 6.4 of the *Code of Standard Practice* outlines basic fabrication tolerances for hot-rolled shapes, and AWS D1.1 addresses basic fabrication tolerances for welded built-up members. Fabrication tolerances for other materials are stipulated in several specifications and codes, each applicable to a specific material in a specialized area of construction. The designer should become familiar with these tolerances and provide adjustment in details to accommodate them. In addition, the steel detailer should prepare the shop drawings consistent with the fabrication standards, design details and tolerances noted.

7.4 COLUMN SPLICES

There is an optimal height for the location of column splices. If the splices are too close to the floor, the splices will not be high enough to allow the OSHA Subpart R mandated safety tie-off attachments for exposed floor edges at the periphery of the building or at interior floor openings. If the splices are too high above the floor, the connectors, bolters and welders will not be able to work comfortably without scaffolding or floats (Figure 7-10).

Design the column splices at a safe working level above the top of the steel. This height is usually in the range of 4 ft above the top of steel. Consider requests from the erector to increase or decrease the designated column splice height.

To avoid column splices that do not line up, provide simple column splice details. Use standard AISC column splice details whenever possible.

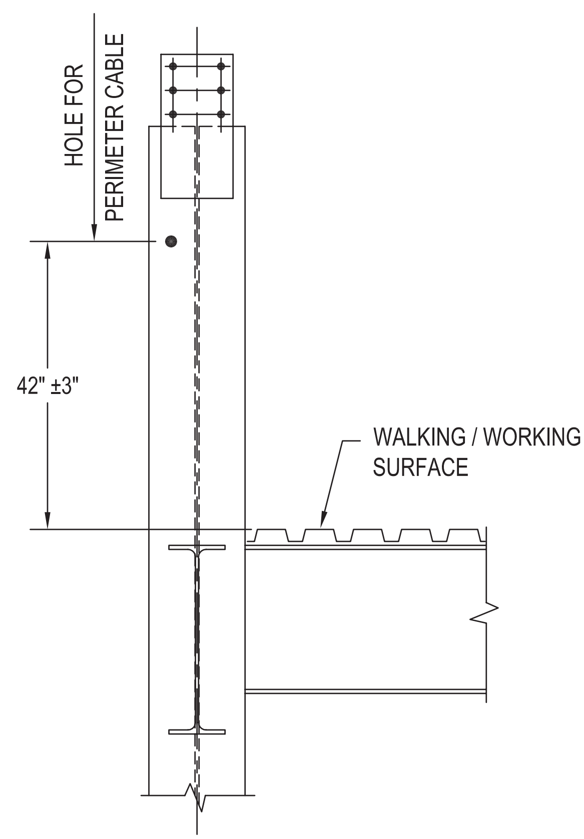

Fig. 7-10. Perimeter column detail allowing for proper location of safety tie-off attachments.

7.5 FAÇADE ATTACHMENTS

Even with all of the AISC tolerances met, there can be problems with fit-up of non-structural steel elements. The SER should investigate accumulated tolerance issues, assist the architect in selection of a fascia connection system and provide adjustable connections where needed. Refer to AISC Design Guide No. 22, *Façade Attachments to Structural Steel Frames* (Parker, 2008).

7.6 HIGH-STRENGTH BOLT USAGE

Bolt uniformity is imperative. Minimizing the number of diameters and grades of high-strength bolts on a given job lessens the chance for a mix-up in the shop or field. Tightening methods as defined in the RCSC *Specification* vary in the ease of application, cost of inspection, and cost of installation.

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