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Preassembly of Structural Systems: Methodology and Application

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PREASSEMBLY OF STRUCTURAL SYSTEMS: METHODOLOGY AND APPLICATION

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Abstract

This report presents the methodology and results of the research performed on the preassembly of structural systems, at the Center for Advanced Technologies for Large Structural Systems (ATLSS) at Lehigh University. The objectives of this research are (1) to identify opportunities for the use of preassembly or prefabrication methods, (2) to develop ways to effectively determine the usefulness of these methods, and (3) to develop conceptual designs of innovative structural systems explicitly for preassembly to test the methodology.

The research focuses on identifying the critical design and construction factors that influence the feasibility and net benefits of a specific preassembled or prefabricated project, as well as developing a methodology to quantitatively determine the possibility of success or potential savings for a given preassembled project. In addition, with the insights gained from detailed site observations of preassembly methods and help from structural designers, we develop and analyze conceptual designs for innovative structural framing systems, which we use to test the methodology. Future research may develop these design concepts in more detail, demonstrate them in the lab and in the field, and work toward their implementation in practice.

Through performing the research described in this report, we conclude that, when properly implemented, preassembly offers the potential to substantially improve construction duration and worker safety. The critical factor measures that we identify provide an objective way to determine the potential advantages and disadvantages of a particular preassembled system, and the methodology that we develop for estimating construction duration and worker air time offers a quantitative way to identify the possibility of success or potential savings of a specific preassembled project. In addition, the conceptual designs of innovative structural systems that we develop provide a way to test the usefulness of the critical factor measures and the methodology, and it is hoped that these concepts may lead to improvements in the economic efficiency and construction effectiveness of preassembly methods.

Chapter 1: Introduction

1.1 Objectives and Scope

This research is motivated by the belief that erection costs, erection schedules, and worker safety can be improved significantly through the application of non-traditional fabrication and erection methods, specifically the preassembly or prefabrication of structural and other facility systems. The objectives of this research are (1) to identify opportunities for the use of preassembly or prefabrication methods, (2) to develop ways to effectively determine the usefulness of these methods, and (3) to develop conceptual designs of innovative structural systems explicitly for preassembly to test the methodology.

The first phase of the research identified the major benefits and difficulties associated with modular construction methods (ATLSS Report 94-11). This second phase of the research focuses on identifying the critical design and construction factors that influence the feasibility and net benefits of a specific preassembled or prefabricated project, as well as developing a methodology to quantitatively determine the possibility of success or potential savings for a given preassembled project. In addition, with the insights gained from detailed site observations of preassembly methods and help from structural designers, we develop and analyze conceptual designs for innovative structural framing systems, which we use to test the methodology. It is hoped that these new design concepts will improve the economic efficiency and construction effectiveness of preassembly methods. Future research may develop these design concepts in more detail, demonstrate them in the lab and in the field, and work toward their implementation in practice.

The proposed research is limited to low and mid-rise buildings up to ten stories, and considers both occupied (e.g., commercial and residential) and industrial (e.g., petro-chemical plant and manufacturing) buildings. The non-traditional fabrication and erection methods considered by the project include prefabrication and preassembly, although preassembly is our primary focus. Prefabrication is a manufacturing process that joins materials into a component part of the final installation, and preassembly is a process that joins materials and prefabricated components (and any equipment) together, at an on-site location away from the final point of assembly, for subsequent installation.

1.2 Organization of Report

This report consists of seven chapters. In Chapter 2, we describe our research approach, explaining the background literature which our research builds upon and the theoretical framework for our work. In Chapter 3, we describe our research methodology, which consists of a combination of field studies, a literature review, telephone interviews, and analyses. In Chapter 4, we discuss the ways in which we characterize the differences between traditional construction methods and preassembly methods, through the results of our site observations, the critical factors of structural systems that we identify, and a literature review that we perform. In Chapter 5, we describe the concepts for three new systems that we develop to take advantage of the inherent benefits of preassembly and prefabrication, as well as a series of analyses we perform on the new systems. In Chapter 6, we discuss the results of our research, and in Chapter 7 we make some general conclusions.

1.3 Conclusions

Through performing the research described in this report, we conclude that, when properly implemented, preassembly offers the potential to substantially improve construction duration and worker safety. The critical factor measures that we identify provide an objective way to determine the potential advantages and disadvantages of a particular preassembled system, and the methodology that we develop for estimating construction duration and worker air time offers a quantitative way to identify the possibility of success or potential savings of a specific preassembled project. In addition, the conceptual designs of innovative structural systems that we develop provide a way to test the usefulness of the critical factor measures and the methodology, and it is hoped that these concepts may lead to improvements in the economic efficiency and construction effectiveness of preassembly methods.

Chapter 2: Research Approach

In this chapter, we describe our research approach, explaining the background literature which our research builds upon and the theoretical framework for our work.

2.1 Background

Previous research by DeLaTorre et al [1994], Tatum et al [1987], Consalvi [1995], and others shows that there are several possible advantages that may result from using special construction methods (preassembly, prefabrication, or modular construction), as well as many possible disadvantages. This section describes these advantages and disadvantages, as well as some guidelines for the use of special construction methods. In addition, we cite some examples of specific projects using special construction methods to illustrate different possible applications of preassembly, prefabrication, and modular construction, and to show what forces prompted their use.

Before discussing the previous research, it is necessary to define the terms that we use in this section. For our purposes, *special construction methods* include *preassembly*, *prefabrication*, and *modular construction* [Tatum et al, 1987]. *Prefabrication* refers to a manufacturing process that joins materials into a component part of the final installation, while *preassembly* refers to a process that joins materials and prefabricated components (and any equipment) together, at an on-site location away from the final point of assembly, for subsequent installation. *Modular construction*, or *modularization*, refers to the incorporation of the equipment, piping, steelwork, instruments, electrical, and other components in the units [Consalvi, 1995]. Consalvi [1995] used the term *segmental construction* to refer to portions of a structure where steel framing is site-assembled at grade and lifted into place (it may or may not contain equipment, etc.), which is similar to our definition of preassembly. For our research, we are mainly interested in the structural members and decking, not in the equipment, piping, instruments, or electric services; therefore, we are concentrating on preassembly and prefabrication of structural systems more than modularization. However, many of the insights associated with modularization also apply to preassembly and prefabrication.

2.1.1 Advantages and Disadvantages of Special Construction Methods

Using a literature study and a survey of 31 companies representing a wide variety of construction types (e.g., bridge, industrial, light industrial/commercial, prison, residential, ship construction) and many points of view (e.g., fabricator, project manager, architect, structural engineer, engineer, erector, and manufacturer), DeLaTorre et al [1994] studied modular construction practices to identify broad advantages and disadvantages over conventional construction practices, as well as differences between modular and conventional construction. Some of the advantages and disadvantages of modular construction they identified are shown in Table 2-1. They found that modular construction differs from conventional construction due to the interdependency of activities with modular construction, which leads to greater complexity and involvement, as well as the fact that many activities are performed earlier in the project and involve increased effort. The driving forces which motivated the companies to use modularization include site resource constraints, reduced cost, reduced schedule, improved safety, and combinations of these.

Table 2-1: Advantages and disadvantages of modular construction [DeLaTorre et al, 1994].

Advantages	Disadvantages
•Reduced cost •Increased quality •Improved safety •Reduced schedule •Reduced social and environmental impacts •Increased possibility of construction	•Need for additional material •Need for additional construction effort •Need for additional coordination of activities •Increased cost •Increased risk •Reduced adaptability to design changes

In a study by Tatum et al [1987], forces prompting the consideration of special construction methods, as well as project implications of these methods were identified. General forces prompting the use of special construction methods are shown in Table 2-2. Project implications of these methods for the overall project are listed in Table 2-3, while implications identified for certain functional activities are shown in Table 2-4. They also studied processes used to evaluate and implement special construction methods and found that they were highly project specific, ranging from systematic studies of feasibility, cost, and schedule to quick decisions based on intuition and judgement, depending on the size of the project and the potential impact of the method.

Table 2-2: General forces prompting the use of special construction methods [Tatum et al, 1987].

•Adverse site and local area conditions •Competitive conditions •Specialized building or process technology •Advantages of manufacturing conditions •Demanding schedule •Owner or regulatory demands •Specialized design requirements •Modular design or repetitive units •Potential cost savings

Table 2-3: Project implications of special construction methods for the overall project [Tatum et al, 1987].

•Increased project feasibility and risk •Changes in project organization •More project planning and progress monitoring •Greater project coordination •Project results such as shorter project schedules, decreased impact on the local area, and significant cost savings

Table 2-4: Implications of special construction methods for certain functional activities [Tatum et al, 1987].

Functional Activity	Implications
Design scope and activities	•Altered division of responsibility •New design criteria •Earlier required decisions •Increased interdependency between design and construction
Procurement operations	•Increased scope and new criteria for decisions •Altered sequence and delivery schedule •Required alternate warranty provisions •Increased difficulty of bid evaluation •Additional shop supervision and quality control
Fabrication, transportation, and construction operations	•Altered division of responsibility •Altered location, sequence, methods, and controls for fabrication •Additional transportation and handling requirements for large assemblies •Altered scope, methods, and sequence of site activities
Testing and start-up operations	•Shortened testing and start-up at the site due to partial testing at the fabrication facility
Facility operation	•Altered structure or functional capabilities

Advantages of modular and segmental construction noted by Consalvi [1995] are listed in Table 2-5, while disadvantages of these methods are listed in Table 2-6. Consalvi also noted that there may be additional complexities with modular and segmental construction; these are listed in Table 2-7.

Table 2-5: Advantages of modular and segmental construction [Consalvi, 1995].

General advantages	•Improved site safety •Reduced interruption to existing operations •Reduced site construction time •Earlier production and market entry •Reduced or eliminated scaffolding and site painting •Eliminated temporary handrail and cable
Advantages of modular construction	•Quality control •Shop effectiveness, select labor pool, quality control in a controlled environment •Reduced number of contractors on site •Reduced congestion and outside labor on site •Reduced start-up cost •Reduction in construction support facilities such as laydown areas and staying areas
Advantages of segmental construction	•All elevations of steel can be ground assembled complete with grating, toe plate, handrail, etc. •Personnel on the ground can move about more safely and iron workers' exposure to falls is eliminated •Gain in efficiency in erection, bolt up, installation of catwalk, handrail, etc. at grade •Sections may be assembled in the fabrication shop if they can be easily transported

Table 2-6: Disadvantages of modular and segmental construction [Consalvi, 1995].

•Possible need for additional site preparation •Possible need for taller and larger crane capacity •Increased transportation risk •Additional construction management cost •Additional engineering hours (transportation, lift, special connections, etc.) •Additional steel and construction cost

Table 2-7: Additional complexities with modular and segmental construction [Consalvi, 1995].

•Additional laydown area and site preparation •Additional supervision and engineering at the site •Location of modular yard and shipping routes •Site clearance for land transportation and erection •Crane capacity and site preparation for the crane •Lifting and rigging methods (size, weight of the module, slope of the road) •Land route to the site •Road limitations •Barges (drift and tidal changes, dock areas)

2.1.2 Guidelines for Use of Special Construction Methods

Consalvi [1995] also established guidelines to help evaluate or plan the use of modular or segmental construction to determine if this type of construction can improve a project and how it may affect project planning, design, and construction. General considerations for modular and segmental construction are listed in Table 2-8.

Table 2-8: General considerations for modular and segmental construction [Consalvi, 1995].

General considerations for modular construction	•Weight and size limits for transportation •Limitations imposed by lifting, jacking, or skidding into position •Effect of a change in weight or arrangement of the center of gravity of the module •Dynamic forces for land and sea transportation •Excavation and site preparation for the modular base •Temporary bracing and tie down steel for shipping the equipment •Load combinations for shipping •Trailer size and support details •Location of utilities
Project specific structural considerations for modular construction	•Modular size and weight limitations •Center of gravity of the module •Modular rigging points and limitations •Pad eyes and lifting lugs •Modular erection sequence •Modular to modular connection sequence •Coordination with heavy lift contractor •Temporary steel bracing members for shipping •Bolt-on ladders, platforms, etc. •Transportation loads (trailer and shipping) •Jacking information •Lifting
General considerations for segmental construction	•Establish process arrangement and bay sizes early without changes •Determine what equipment is to be attached before erection •Consider the layout area for segments •Determine framing arrangements and the direction of interior framing early •Carefully work out segment layout, splice location, and connection details •Establish bracing arrangements early •Include permanent bracing in module to stabilize during erection •Determine the center of gravity of the segments •Detail pin-guide connections •Consider size and availability of the crane

2.1.3 Examples of Projects Using Special Construction Methods

In this section, we describe some examples of specific projects using special construction methods to illustrate different possible applications of preassembly, prefabrication, and modular construction, and to show what forces prompt their use. Although the scope of our research is limited to the preassembly of typical low to mid-rise buildings, examples of other types of projects employing special construction methods provide an idea of the extent of what is possible and what types of problems may be encountered.

Rapid construction is the motivation for the use of precast concrete modules for building jailhouses [Tarricone, 1991]. While the site is being prepared and the foundation is being poured, the modules are constructed in the factory, dramatically reducing construction time. The modules are three-dimensional precast concrete cubes, complete with the bed, sink, and toilet cast into the concrete. For a four-story facility in Virginia, it took only five weeks to erect the 176 cells. Another advantage of this type of construction is the quality control available with factory production, although this process may not reduce overall costs, and depending on location, transportation costs may become substantial.

Modular steel bridges may be an effective alternative to conventionally constructed bridges for bridge replacements or permanent structures [Shaker and Greenwald, 1994]. Material costs for the modular steel bridges are similar to or slightly higher than traditional steel and concrete bridges; however, the quick erection and potential cost savings from engineering, erection time, labor requirements, and maintenance and repair, make modular steel bridges a viable option. The Bailey bridge, the first bridge of this type, was introduced for quick deployment during World War II; a typical Bailey bridge can be completely installed in a matter of days. Now there are many other companies producing modular steel bridges, with improved properties, quicker assembly, and configurations that accommodate various span and capacity requirements.

The Modified Roof Erection System (MRES), developed by James N. Gray Construction Company, is a roof-erection process which offers potential benefits over conventional steel roof erection, such as increased safety, higher productivity, less impact from skilled labor shortages, shortened construction schedule, better quality, and reduced cost [Stevens and Murray, 1994]. The process involves assembling roof modules at a level near the ground and then hoisting them into place. Another benefit, in addition to those mentioned, is the opportunity for electrical, mechanical, and fire-protection trades to install their materials at the low level. Possible risks of this process include the possibility of increased costs and the risk of physical damage during the lifting operation. A successful trial of the MRES was performed on 12 two-bay (50 *ft* by 60 *ft*) modules.

For the Don Valley Parkway/CP Rail Grade Separation in Ontario, engineers designed a 2,200 *ton*, 105 *ft* long reinforced concrete arch that was constructed next to a railway and jacked 92 *ft* from its constructed position to its final position beneath operating railway tracks [Anderson, 1990]. The motivation for using prefabrication in this case was the inability to interrupt train schedules and automobile traffic during construction of the arch. The arch was to be part of a new ramp to a busy highway, underneath a heavily travelled railway line.

Construction of the expansion of New York Hospital in Manhattan, which spans the six-lane Franklin Delano Roosevelt Drive, was made possible with the use of preassembly and prefabrication techniques [Brazil et al, 1995]. Severe site restraints, such as a small construction zone, limited crane use, restrictions on the weight of construction equipment, and limited hours when the roadway could be closed, forced the use of non-traditional construction techniques. A 485 *ft* long platform, which supports the 12-story hospital addition, was constructed from 20 prefabricated steel trusses with infill framing between them which were preassembled as box units. The platform panel sections, which were typically 91 *ft* wide by 51 *ft* long and weighed up to 760 *tons*, were transported and preassembled on barges and erected with barge-mounted cranes.

These examples of practical applications of preassembly or prefabrication methods show that there are always forces or intended benefits that motivate the use of these methods, such as faster construction, increased safety, better quality, or special circumstances when traditional construction methods may not work. There are also usually ways in which the use of these methods affects the project negatively, such as potentially higher costs or increased risk of damage to the structural components. These benefits and implications were noted in the studies by DeLaTorre et al [1994], Tatum et al [1987], and Consalvi [1995], as discussed in section 2.1.1. However, a study of diverse case studies such as these will not lead to methods of identifying the possibility of success or potential savings of a project. A systematic methodology is needed to accomplish this; section 2.2 describes the theoretical framework of our research and our approach to meeting these objectives.

2.2 Theoretical Framework

As mentioned in the background literature, previous research shows that there are several possible benefits that can result from using special construction methods (preassembly, prefabrication, or modular construction), as well as many implications. Many companies decide to use special construction methods because of certain driving forces, also mentioned in the literature. Our purpose is to identify how the critical factors of the construction process (e.g., fabrication efficiency, transportation capacity, or erection safety) are affected by the characteristics of the project (e.g., the size or weight of the preassembled sections or the number of units to be preassembled). Once these interactions are identified, it is possible to identify which project characteristics may maximize the benefits and minimize the negative implications, and which tradeoffs must be made between the critical factors to optimize the overall project, since it is recognized that every critical factor cannot be optimized simultaneously. Based upon this analysis, we develop concepts for three new systems designed explicitly for preassembly in response to the critical factors.

Our focus is on low to mid rise commercial, residential, or light industrial buildings, since this type of building has the largest market share. We are mainly interested in the structural members and decking, not in the equipment, piping, instruments, or electric services; therefore, we are concentrating on preassembly and prefabrication of structural systems more than modularization, although we include planar and three-dimensional sections in our study.

As described in section 2.1.2, Consalvi gave basic guidelines for the use of special construction methods, as well as issues that must be considered when using these methods. For example, Consalvi noted that there are certain weight and size limitations of a module and that dynamic forces for land and sea transportation must be taken into account. With our approach, we describe which aspects of the project (critical factors) are affected by certain project characteristics (e.g., size, weight, number of units, etc.), and whether they are affected positively or negatively. This provides an objective, but unscaled method to determine whether a given system may provide advantages over traditional construction.

In developing the new systems, we respond to the insights gained from the development and study of the critical factors. For example, we attempt to reduce the redundancy of members, to use standard forms, and to perform decking activities on the ground. Our ultimate goals in developing these new systems are to reduce the erection time, the amount of time workers spend in the air, and the overall cost while maintaining the structural integrity of the system. It should be noted that for the development of the new structural design concepts, we concentrate on preassembled systems rather than prefabricated systems, due to the many transportation issues associated with prefabrication.

Chapter 3: Research Methodology

This chapter describes the methods we use to conduct our research. We use a combination of field studies, a literature review, telephone interviews, and analyses to develop our conclusions.

3.1 Site Observations

Observations of construction site activities, specifically traditional methods juxtaposed with preassembly methods, provide a unique opportunity to directly compare the two methods. Such a situation was afforded by DuPont's facility in Johnsonville, TN, the primary site for direct observation of these methods. A special building was designed for a series of new production activities within the operating facility. The structural design of the facility explicitly considered and included the preassembly of several portions of the structural steel and incorporated a new connection (the ATLSS Connector) into two specific preassembled sections. These site observations of traditional construction and preassembly not only provide information on the nature and flow of the activities for the two types of construction, but also provide production data for the two methods for a side-by-side comparison.

Our research activities include analysis of the structural framing system with predictions on the nature, sequence, and duration of each construction activity for the structural steel erection. We then directly compare these predictions to site activities in extensive observations (almost 70 labors hours of observation). We then use the major attributes of the activity flow and the observed production rates to revise the methodology for prediction and provide a basis to identify the critical criteria in design and construction affecting the duration, cost, and safety of structural steel erection.

We discuss the results of the site observations in section 4.1. These results include a comparison of the production rates of traditional and preassembled structural units, the methodology that we develop for estimating project duration, and some general conclusions that we draw from the site observations. In section 4.2, we discuss the critical factors for structural systems that affect the duration, cost, and safety, that we identify based on the site observations and data.

3.2 Literature Review

The use of standard structural elements, one of the critical factor measures we identify, seems to be extremely important to the ability to implement structural systems. Therefore, before we attempt to develop the new framing system concepts discussed in section 5.1, we research through a comprehensive literature review the types of structural forms and connectors that are available, the types of structural elements and connections that are available, the systems that currently exist, and new ways of thinking about the combination of structural elements. We focus on steel and precast concrete forms, since these materials can be used for preassembly and prefabrication of structural systems.

We describe this basis for the new framing systems in detail in ATLSS Report No. 95-10, "Components of Structural Systems in Steel and Precast Concrete," [Farschman and Slaughter, 1995]. This report contains a compilation of existing standard, nonstandard, and proprietary structural forms, structural connectors, load bearing elements, and connections using steel or precast concrete. It focuses on the superstructure of buildings, including the connection of the superstructure to the foundation. It also includes an analytical framework of structural elements and connections that represents all of the feasible combinations of the different types of structural elements. We also identify some examples of structural systems specifically designed with complementary elements and connections.

We discuss the results of this literature review in more detail in section 4.3.

Using the site observations and the standard structural elements, including the critical factors for structural systems, we develop concepts for three new structural systems designed explicitly for preassembly. We develop these new systems, described in section 5.1, in response to the critical factors and from insights gained from the site observations.

3.3 Interviews

In order to get an idea of the technical feasibility of the new systems, in terms of structural capacity, structural stability, and performance, as well as the validity and comprehensiveness of the critical factors, we want feedback from experts in the field, such as structural designers, fabricators, manufacturers, and erectors. We sent descriptions of the new systems and the list of critical factors and measures to several people from industry, including the members of the Structural Assemblies Advisory Group for the ATLSS Research Center at Lehigh University, as well as some Lehigh University professors. The names and companies of the people who were particularly helpful, by providing the most feedback about the new systems and other preassembly issues, are listed in Table 3-1, and the addresses and phone numbers of all the people we contacted, including those who referred us to other contacts, are in Appendix A. Through telephone interviews, we get their impressions of the new systems, including their perceptions on the technical feasibility of the systems, possible project impacts in terms of duration, cost, safety, and performance, comments involving the details of the systems, and suggested improvements. We respond to this feedback by modifying some of the aspects of the systems. We also get their impressions of the critical factors, which help us to improve the critical factors and make them clearer and more understandable. We discuss some of their general comments and concerns in the discussion in Chapter 6.

Table 3-1: Industry members interviewed.

Industry member	Company
Robert G. Abramson	Interstate Iron Works Corporation
Kathleen Almand	Civil Engineering Research Foundation
Steven J. Bianculli	U. S. Steel
Bob Dunn	National Riggers & Erectors
Milton C. Gore, Jr.	DuPont Engineering
Richard H. Hendricks	DuPont Engineering Center
Robert Holliday	Benham Group
Timothy L. Horst	Bechtel Corporation
Nestor Iwankiw	American Institute of Steel Construction
Kazuhiko Kasai	Lehigh University
Raymond W. Monroe	Steel Founders' Society of America
Brett Paddock	Falcon Steel Company
Pravin Patel	DuPont Engineering
Tom Schlafly	American Institute of Steel Construction
J. H. (Ted) Temple	Chaparral Steel Company
Andrew Ziolkowski	American Iron & Steel Institute

3.4 Analyses

In order to determine whether the new systems might be technically feasible and improve project results, we perform a series of analyses on the new systems. For each of the new systems, we perform analyses with respect to the critical factors, structural performance, and project impacts. We also perform a project impact analysis for a traditionally constructed building to provide a basis for comparison. These analyses give us ideas as to whether the systems are effectively designed for preassembly, whether they are structurally sound, and whether their implementation may improve cost, safety, or duration.

3.4.1 Analysis with Respect to Critical Factors

Before we perform a structural performance analysis and project impact analysis, we perform an analysis of each system with respect to the critical factors and their measures to determine whether the systems are effectively designed for preassembly. This analysis provides insight into which critical factors are increased when compared to traditional construction and which critical factors are decreased. In order for the system to be provide an overall advantage, the benefits to the critical factors must outweigh the disadvantages. However, in extreme cases when it is especially important to increase the benefits of a particular critical factor, such as erection duration, for example, even outstanding disadvantages incurred to the other critical factors may be acceptable.

We use this analysis to determine the most promising variations of each of the new systems. For each variation of each new system, we go through the measures of the critical factors to identify how the characteristics of the system affect each of the critical factors, whether positively, negatively, or not at all. This process brings out all the issues that must be considered for each system.

We describe the results of these analyses for each system in Chapter 5 (sections 5.2.2, 5.3.2, and 5.4.2).

3.4.2 Structural Analyses

We perform a basic structural analysis for Systems 1 and 2 to determine whether the buildings have the capacity to withstand basic gravity loads, using a specific design for a prototype building. (Due to the complexity of System 3, a structural analysis of this system is beyond the scope of this report.) The live and dead loads applied to the prototype building are those for a typical office building. The Load and Resistance Factor Design (LRFD) method is used with the load combinations which involve only gravity loads.

The analysis procedure involves calculating the required capacities of the different structural elements (e.g., girders, columns, connections) and determining whether the available structural members (e.g., channels, concrete-filled tubular columns) have sufficient strengths for use in the specified configurations. The structural analyses that we perform are very basic to get an idea of whether the systems offer the potential to work and to obtain the approximate sizes and numbers of members that could be used for each system, as well as the approximate connection types. We analyze the systems considering only gravity loads, assuming that an appropriate means of resisting lateral loads (e.g., wind and seismic loads), such as bracing members or additional reinforcement of connections, would be provided. We use the information we obtain from the structural analyses in the project analyses, which we discuss in section 3.4.3, to calculate the approximate duration for construction of each building, as well as worker air times. Actual testing must be done before any of these systems can be put into practice.

We discuss the results for the structural analyses of Systems 1 and 2 in Chapter 5 (sections 5.2.3 and 5.3.3), and the detailed structural analysis procedure (with calculations) is described in Appendix D.

3.4.3 Project Analyses

After performing the structural analyses which result in the approximate sizes and number of members and connection types for each system, we perform project analyses to determine the cost and duration for construction of each system, as well as worker air time. From this information, we compare the systems to determine which are most advantageous in the areas of cost, duration, and safety.

We develop the methodology for estimating project durations of partially preassembled buildings from the site observations and data collected from the DuPont construction project, using production rates which factor in the critical design and construction criteria. This methodology involves using a large spreadsheet to calculate the required time for each individual activity involved in the construction process and to combine these activities and calculate the total elapsed time for construction. The information required for this process includes sizes and numbers of members, panels, and connections, as well as an idea of the project's flow.

The primary factor we use to quantify worker safety is the estimated time spent by the workers in the air. From the information calculated from the duration analysis, we simply calculate the amount of time for activities where workers are in the air for each of the new systems and compare this to the worker air time for a traditionally constructed system. This forms the means of comparison between the systems for worker safety.

We discuss the results of the project analyses for Systems 1 and 2, including duration and safety impacts, in Chapter 5 (sections 5.2.4 and 5.3.4), and we describe the detailed project analysis procedure in Appendix E.

Chapter 4: Comparison of Traditional Methods to Preassembly

In this chapter, we discuss the ways in which we characterize the differences between traditional construction methods and preassembly methods. First we describe the results of our site observations, which include a comparison of the activities involved in the two methods and a methodology that we develop to estimate construction durations. Then we discuss the critical factors of structural systems that we identify that influence whether or not preassembly or prefabrication may be beneficial over traditional construction for a specific project. Finally, we describe a literature review that we conduct to identify the current available structural elements, connections, and structural systems.

4.1 Site Observations

As discussed in section 3.1, the observations of construction site activities at DuPont's facility in Johnsonville, TN provide insight into the nature and flow of the activities of preassembled and stick-built construction. The observations also provide production data for the two methods for a side-by-side comparison. This section describes the conclusions that we make from the direct site observations and data obtained at the site.

The first part of this section summarizes the key production rate differences between traditional and preassembled structural units, and, within the preassembled units, between the vertical and horizontal units. Then we describe the methodology for estimating project duration, through an explication of the general procedure and the application of the methodology to the two building portions for the DuPont project. Finally, the last part of this section describes some general conclusions that we draw from the site observations.

4.1.1 Summary Comparison of Traditional Steel Erection to Preassembly Methods

The direct comparison of the traditional ("stick-built") method of structural steel erection and the new method of preassembly provides significant insights. Table 4-1 is a summary of the key differences in the construction activities between the methods. This table compares traditional construction activities to preassembly activities for both vertical units and horizontal units. For our purposes, a *bent* refers to a vertical preassembled unit, and a *panel* refers to a horizontal preassembled unit. As noted often in Table 4-1, the differences in relative time between the bents, panels, and stick-built members depend on many other factors, including the size and complexity of the unit, as well as the number of connections that connect the unit to the rest of the building. These observations lead to the development of the detailed critical factors we discuss in section 4.2.

While several of the materials handling activities are unchanged by the introduction of the preassembly methods (e.g., unloading a truck and shaking out the members), major differences do exist in the amount and nature of preparation for the two methods and in the performance of the activities. The preassembly of the structural units on the ground allows the workers to move about freely, without the danger of falling from a height, and within relatively close proximity to the tools and materials. In contrast, the traditional method requires that the workers transport the tools and materials to the above-ground location and then back to the ground when the task is complete. The preparation of the site and the layout of the structural members for the preassembled unit take more time than preparation for the traditional method because preassembly precludes the placement of the members in their final location and therefore alignments and relative positioning are more crucial. In addition, the lifting and positioning of the preassembled units is more complex than the comparable activities for a single member. The time required for the permanent connection of the preassembled sections, however, is significantly less than it is for the traditionally erected members.

4.1.2 Methodology for Estimating Project Durations

This section contains the methodology for estimating project durations using preassembled structural elements and traditional methods, using production rates which factor in the critical design and construction criteria. The methodology is currently enhanced through the application of a commercially available computer spreadsheet program (Quattro Pro). Using the methodology, the duration is estimated for two sections of the DuPont project, and the estimated duration predicts the actual duration to within 80% for the first portion (Building A) and to 100% for the second portion (Building B).

4.1.2.1 Procedure for Estimating Duration and Worker Air Time

We develop the procedure shown in Table 4-2 to estimate the duration of a partially preassembled construction project. In order to carry out this procedure, we need the structural plans of the building, as well as an idea of the detailed flow of activities. We create a large spreadsheet to aid in the calculations, with one section to calculate the required time for each major individual activity, and another to combine these activities and calculate the total elapsed time for construction. To get the production rates for each activity, needed in the first section, we use the actual data from the site observations; each individual activity is isolated, and the times for activities that were not observed are interpolated or estimated. This "Average and Interpolated Times of Activities" is shown in Appendix B. In the second section, the Critical Path Method is used to combine the activities in the first section to get the total elapsed time for construction; when two activities occur simultaneously, only the activity of longest duration is included. An example of the spreadsheet is given in Appendix E with the detailed project analysis of the new structural systems described in Chapter 5.

4.1.2.2 Example Application: DuPont Facility, Johnsonville, TN

The construction project of interest at the DuPont facility in Johnsonville, TN, consists of four major parts, as illustrated in the site plan in Figure 4-1, located in column lines 1 through 9, AA through H. The four main parts consist of 37'-1.5" high Building A (column lines 5 through 7.4, ED through H), 58'-7" high Building B (column lines 7 through 9, AA through DJ), 124'-0" high Building C (column lines 1 through 2, AJ through CJ), and an intermediate section connecting the other buildings. We were able to observe some of the construction of the Building B, and we use the procedure we developed to estimate the durations of the construction of this building and Building A. We obtain information on the actual construction durations of the two buildings from the site supervisor.

Summary of Building A

The assumption for the time estimation for this building is that three bents and one roof panel were preassembled, arranged in the layout illustrated in Figure 4-2, and that the intermediate floor and fourth side of the building were stick-assembled. We assume that the flow of activities and time for each activity were similar to those of Building B that we observed.

We estimate the time for construction using the procedure discussed in section 4.1.2.1. We count and categorize the members and connections, depending on which bent or panel they are in or if they are stick-built. Then, we estimate the time for each activity, depending on certain factors. For instance, we calculate the activity duration from the number of members and whether the section is a bent or panel. Some of the factors, such as the tolerances, appear to be very important but cannot be accounted for at this time. After we calculate the time for each activity, we calculate the total estimated elapsed time, based on an assumed flow of activities and critical path, shown in Figure 4-3.

The final estimated time for construction of Building A is 36.9 hours, or 0.8 weeks. The actual time was approximately 45 to 67 hours, or 1 to 1.5 weeks, with a payroll base of 45 hours per week. One possible reason for the difference between the actual and predicted times is that our time estimation is based on the production rates measured for Building B, which occurred after the erector had more experience with the process. In addition, tolerance problems were encountered during the erection of the roof panel due to the fact that the columns were out of plumb. Construction on this portion of the project started on 15 November, 1994.

Table 4-1: Summary of activities involved in using traditional or preassembly methods.

ACTIVITY	BENTS (relative time)	PANEL (relative time)	STICK-BUILT MEMBERS (relative time)
Unloading (all members)	same for all	same for all	same for all
Shakeout (all members)	same for all	same for all	same for all
Assembly (panels, bents), Erection (stick-built)			
Preparing area where section is to be assembled	depends on size of area	depends on size of area	N/A
Workers getting into position	same for preassembled	same for preassembled	plus time to get in air
Getting bolts & tools	same for preassembled	same for preassembled	plus time to lift in air
Getting crane into position	same for all	same for all	same for all
Moving crane to first member	same for all	same for all	same for all
Setting blocks in approximate location	depends on size & complexity of bent	depends on size & complexity of panel	N/A
Hooking member	same for all, depending on member	same for all, depending on member	same for all, depending on member
Lifting member	same for all, depending on distance	same for all, depending on distance	same for all, depending on distance
Measuring/positioning member, adjusting blocks	same for preassembled, depending on member	same for preassembled, depending on member	N/A
Receiving/positioning/aligning member	N/A	N/A	depends on member
Attaching member (2 bolts/end)	same for preassembled	same for preassembled	plus more time (in air)
Unhooking member	same for preassembled	same for preassembled	plus more time (in air)
Crane moving to next member	same for all	same for all	same for all
Workers moving to next location	negligible	negligible	plus more time (in air)
Measuring diagonals, checking levelness, etc.	depends on size & complexity of bent	depends on size & complexity of panel	N/A
Installing remaining bolts	same for preassembled	same for preassembled	N/A
Tightening bolts	same for preassembled	same for preassembled	- N/A

Table 4-1 (cont): Summary of activities involved in using traditional or preassembly methods.

ACTIVITY	BENTS (relative time)	PANEL (relative time)	STICK-BUILT MEMBERS (relative time)
Erection (panels, bents)			
Attaching spreader bar to crane	same for preassembled	same for preassembled	N/A
Moving crane to panel/bent	same for preassembled	same for preassembled	N/A
Attaching spreader bar to panel--balancing	generally less time than panels, depends on size and weight	generally more time than bents, depends on size and weight	N/A
Workers getting into position	less time (if no one in air)	more time (if workers in air)	N/A
Lifting panel/bent into position	generally less time than panels[1]	generally more time than bents[1]	N/A
Aligning panel/bent with connections	depends on tolerances	depends on tolerances	N/A
Connecting panel/bent to building	depends on # of connections	depends on # of connections	N/A
Unhooking panel/bent	generally less time than panels	depends on size of panel	N/A
Moving crane back to ground	same for preassembled	same for preassembled	N/A
Detaching spreader bar	same for preassembled	same for preassembled	N/A
Workers climbing down	no time (if no one in air)	more time (if workers in air)	N/A
Installing guy wires	depends on size, stability of bent	N/A	N/A
Plumbing/leveling (entire building)	depends on tolerances, # conns, & accuracy of preassembly	depends on tolerances, # conns, & accuracy of preassembly	depends on tolerances, # conns
Permanent connection (in air)			
Workers getting into position	same for all[2]	same for all[2]	same for all[2]
Getting bolts, tools to workers	same for all[2]	same for all[2]	same for all[2]
Installing remaining bolts	same for all[2]	same for all[2]	same for all[2]
Tightening bolts	same for all[2]	same for all[2]	same for all[2]
Moving to next connection	same for all[2]	same for all[2]	same for all[2]

[1] This depends on many other factors, including size, weight, and maneuverability.
[2] However, preassembled sections have fewer connections remaining.

Table 4-2: Procedure for estimating the duration of a partially preassembled building.

1) From the structural plans, identify the panels and bents, and for each panel/bent determine the following:
   a) The number of each type of member in the panel/bent. Specify the type, size, and number of ends that are connected during layout and attachment.
   b) The number of each type of connection made during assembly, including the number of bolts per connection, separated between the number of bolts installed during layout and the number of bolts remaining to be installed after layout.
   c) The number of connections to be made during erection of the panel/bent, including the number of bolts per connection, separated between the number of bolts installed during erection and the number of bolts remaining to be installed after erection.
   d) The size and weight of the panel/bent.

2) From the structural plans, identify the stick-built members. Determine the following:
   a) The number of each type of stick-built member in the building (by type and size).
   b) The number of connections to be made during erection of the stick-built members, including the number of bolts per connection, separated between the number of bolts installed during erection and the number of bolts to install after erection.
   c) The number of "sets" of members to be erected and which members and connections are to be included in each set.

3) In the first section of the spreadsheet, to compute the time for each separate activity:
   a) Enter the production rate for each activity (from Appendix B, "Average and Interpolated Times of Activities"), corresponding to the properties of the members, panels, bents, etc.
   b) Enter the number of members, connections, etc. from parts 1 and 2.

4) Using the time for each separate activity calculated in part 3 and assumed flow of activities, sequentially list the activities and compute the cumulative times of the activities. (Use the Critical Path Method--for activities occurring in parallel, only include the activity of longest duration in the cumulative time.)

Summary of Building B

The assumption for the time estimation for this building is that four bents and six panels were preassembled, arranged in the layout illustrated in Figure 4-4, and the rest of the building was stick-assembled. We assume that the flow of activities and time for each activity were similar to the times that we observed.

We use the same procedure for the time estimation of this building as we use for Building A. We estimate the time for construction using the procedure discussed in section 4.1.2.1. We count and categorize the members and connections, depending on which bent or panel they are in or if they are stick-built. Then, we estimate the time for each activity, depending on certain factors. Some of the factors, such as the tolerances, appear to be very important but cannot be accounted for at this time. After we calculate the time for each activity, we calculate the total estimated elapsed time, based on an assumed flow of activities and critical path, shown in Figure 4-5.

The final estimated time for construction of Building B is 180.8 hours, or 4.0 weeks. It actually took approximately 180 hours, or 4.0 weeks, with a payroll base of 45 hours per week, to complete this portion of the project. The reasons for the similarity between the actual and predicted times, when compared to Building A, may be that since the crew had more experience at this point, their production rates were more consistent, and since the columns were larger and the structure was more complex, the erectors recognized the need to plumb the columns before erection of the panel, resulting in fewer tolerance problems. Construction on this portion of the project started on 12 December, 1994.

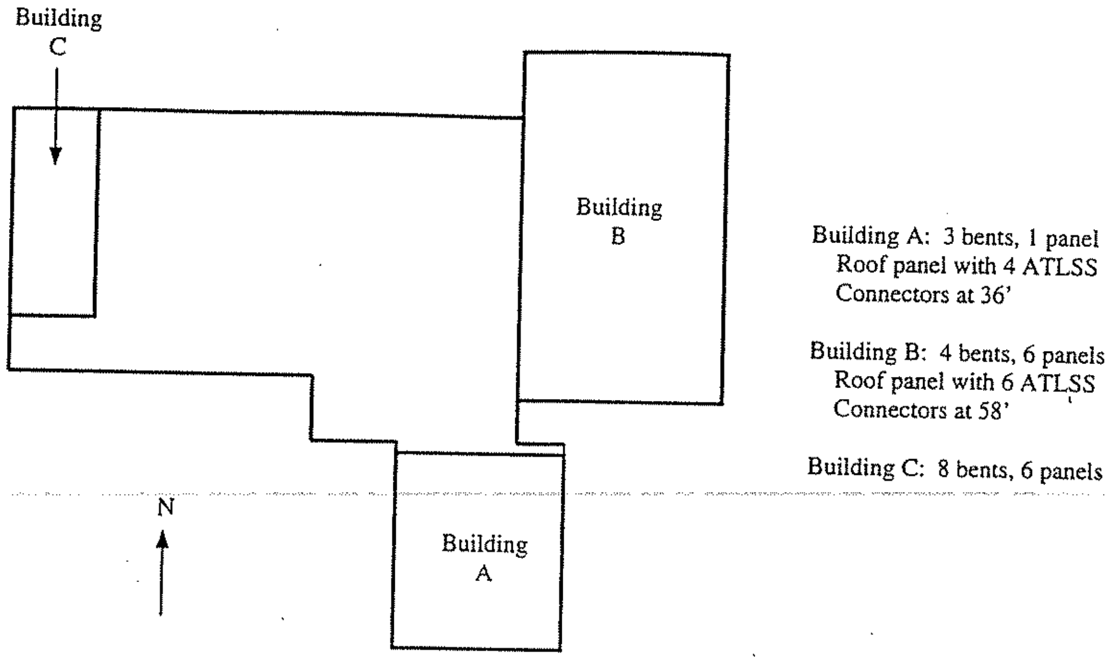


Figure 4-1: Site plan of DuPont facility.

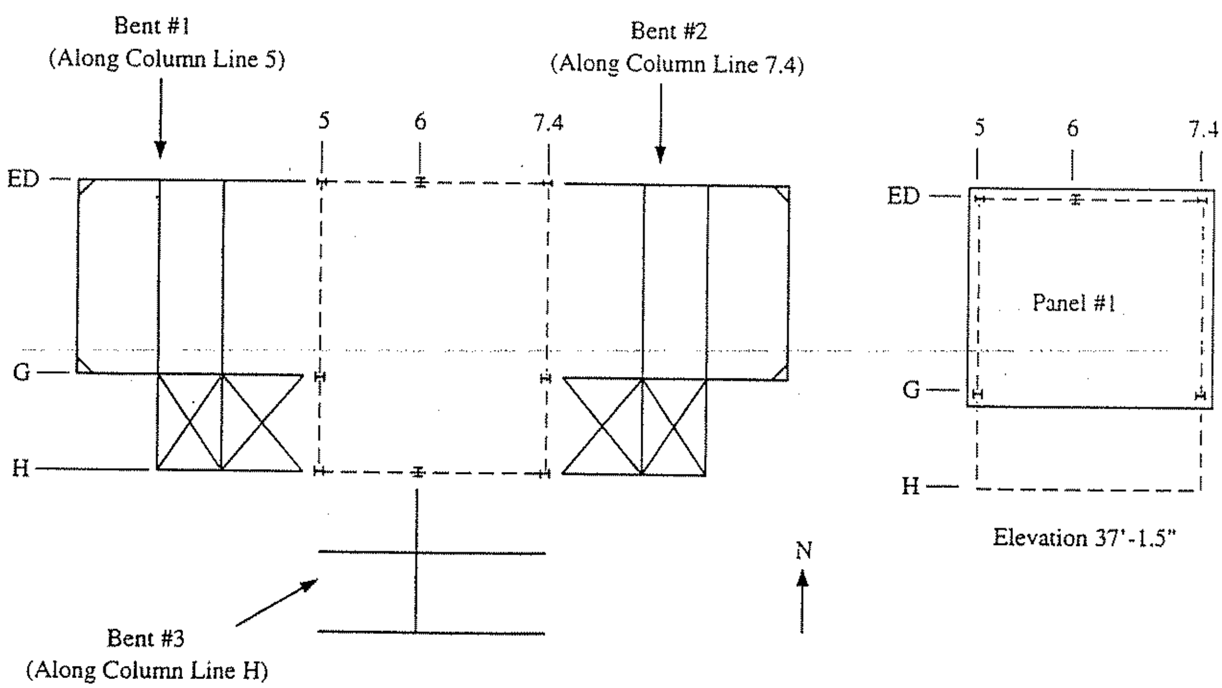


Figure 4-2: Layout of Building A.

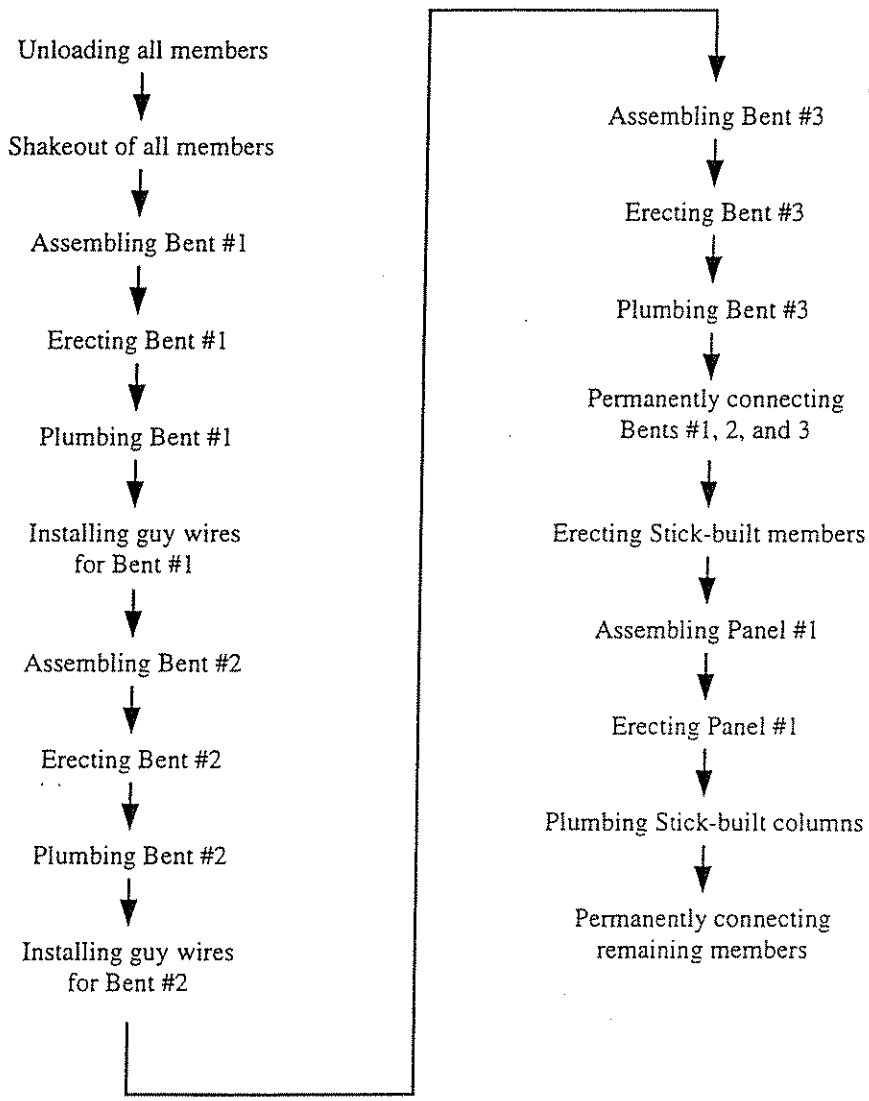


Figure 4-3: Flow of activities for Building A.

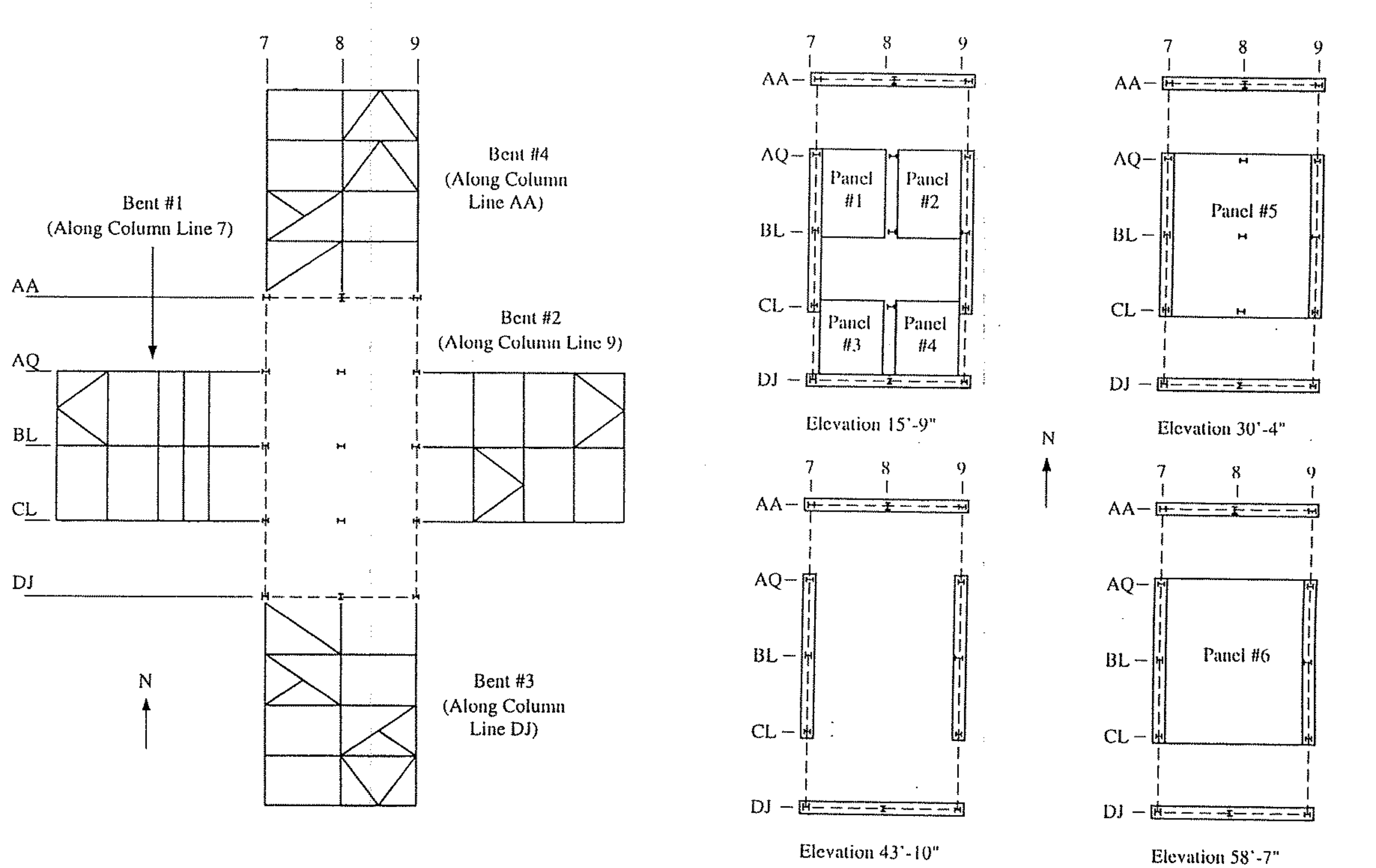


Figure 4-4: Layout of Building B.

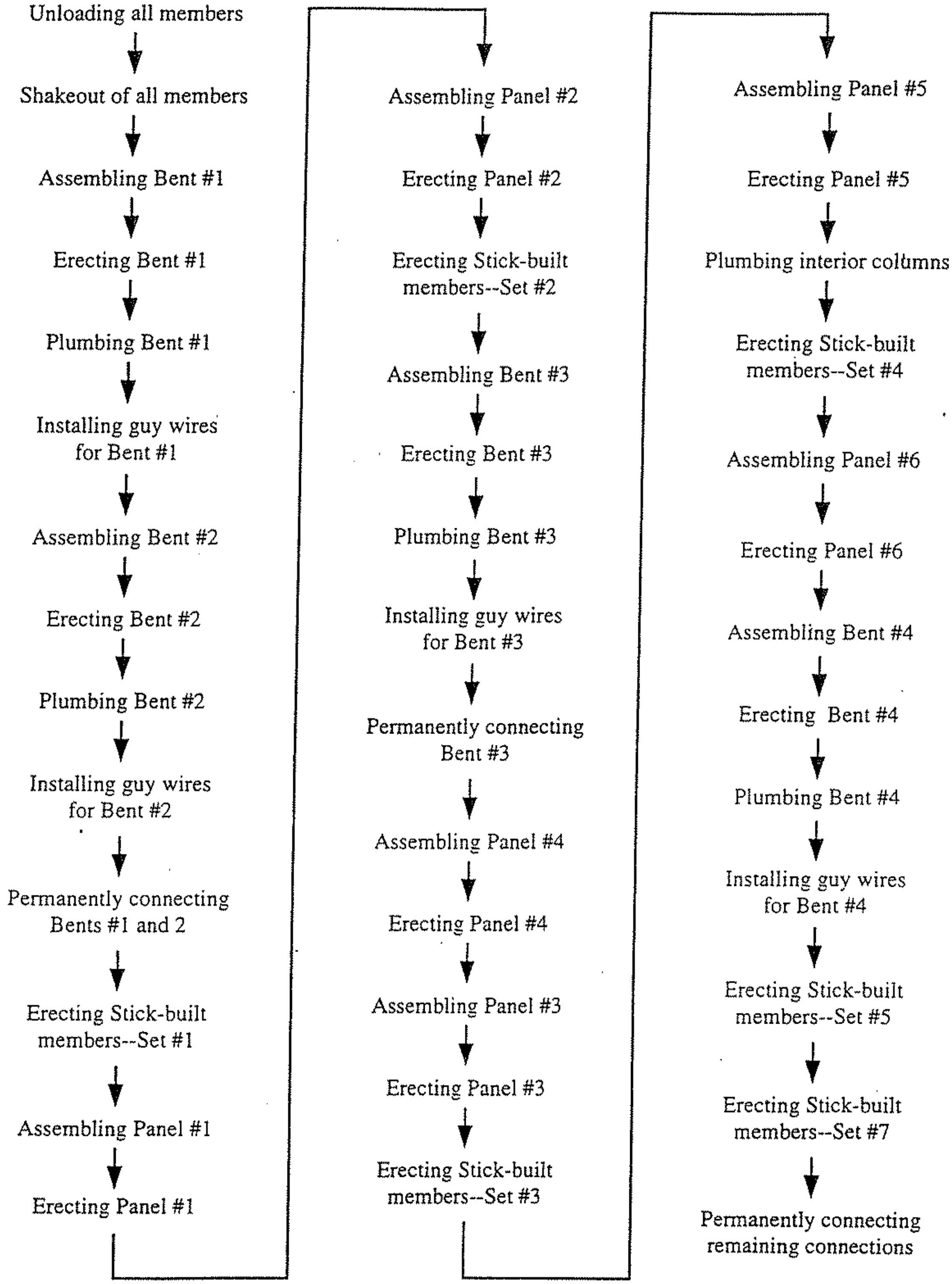


Figure 4-5: Flow of activities for Building B.

4.1.3 Conclusions from Site Observations

Using the methodology that we developed to estimate construction duration and worker air time, we compare the durations and worker air times of the two DuPont buildings, for the actual constructed building (partially preassembled and partially stick-built) and if the buildings were totally stick-built (Table 4-3 shows the percentage of the members in each building that were actually preassembled). Then we calculate the percent of reduction in duration and worker air times due to preassembly (see Tables 4-4 and 4-5). We learn that for Building A, which was 55% preassembled, there is an estimated 8% reduction in duration, and for Building B, which was 75% preassembled, there is an estimated 13% reduction in duration. The difference in savings between the two buildings may be due to the difference in the percentage of preassembled members in the buildings or due to the possibility that the worker productivity for the preassembled buildings followed a learning curve, since Building A was constructed first.

There is also an estimated 47% reduction in worker air time for Building A and an estimated 46% reduction in worker air time for Building B, which indicates that the safety of the workers was improved significantly with the use of preassembly. We also note that certain practices, such as performing decking activities on the ground as part of the preassembled panel and providing an alignment mechanism during erection, might improve the safety and duration of erection even more significantly. We consider these conclusions in the development of the new systems discussed in Chapter 5.

Table 4-3: Proportion of preassembled units for DuPont buildings.

Building	% Preassembled
Building A	55% of members
Building B	75% of members

Table 4-4: Comparison of preassembly to stick-built methods--duration.

Building	Preassembled and stick-built duration	Stick-built duration (Estimated)	% Reduction
Building A	37 hours	40 hours	8%
Building B	183 hours	210 hours	13%

Table 4-5: Comparison of preassembly to stick-built methods--air time.

Building	Preassembled and stick-built air time	Stick-built air time (Estimated)	% Reduction
Building A	18 hours	34 hours	47%
Building B	100 hours	184 hours	46%

4.2 Critical Factors for the Preassembly of Structural Systems

From observations at the DuPont site, it is apparent that the success of the preassembly method (i.e., whether preassembly is beneficial over traditional methods) depends upon a number of factors. We identify these factors and develop a method of determining the issues involved in selecting appropriate levels of preassembly during design and planning. (We include critical factors which we believe apply to prefabrication and transportation, although we do not describe these processes in as much detail as other processes.)

The critical factors that we identify are the efficiency, capacity, performance, and safety of the various activities that take place during the overall process of construction. The specific phases that we identify include design, fabrication, prefabrication, transportation, handling, preassembly, erection, plumbing, permanent connection, and decking/slab. Efficiency, capacity, performance, and safety do not necessarily apply to all of the activities; the applicable critical factors are listed in Table 4-6.

The *efficiency* of an activity refers to the rate of output to input during the activity or how quickly and easily the activity can be carried out. For example, it is generally more efficient for workers to erect a small unit than a large unit. The *capacity* of an activity refers to the amount of material that can be accommodated during the activity. For example, for a given transportation unit, there is a maximum size unit than can be transported. The *performance* refers to how the members or units behave during the activity in response to the conditions and loads imposed on them. For example, a fragile member may not be durable enough to withstand the conditions imposed during handling. The *safety* of an activity refers to the well-being of the workers during the activity. For example, it is generally safer for workers to perform a given activity on the ground than in the air.

For structural erection, we focus on ten distinguishable phases, which can be defined as follows. *Design* refers to the process of choosing structural materials, members, and connections in response to owner requirements and codes in order to create an efficient structure; the design requirements for every project, which must be satisfied regardless of the critical factors, are shown in Table 4-7. *Fabrication* refers to the process of preparing all the individual members for prefabrication, preassembly, or stick-built erection by cutting them to the proper size, drilling necessary holes, doing necessary welding, and attaching any necessary small pieces such as plates, angles, and tees. Members are not joined together during fabrication. *Prefabrication* refers to the process of combining individual members in the fabrication shop to form a larger unit. Welding, attachment of pieces, and installation and tightening of bolts are done to connect different members, but additional holes are only drilled when necessary, such as when an error has been made. *Transportation* refers to the process of moving members or prefabricated units from the fabrication shop or prefabrication shop to the field. *Handling* refers to the process of unloading and shaking out members or prefabricated units from the transportation unit. *Preassembly* refers to the process of combining individual members in the field, away from their final erected position, to form a larger unit. As with prefabrication, welding, attachment of pieces, and installation and tightening of bolts are done to connect different members, but additional holes are only drilled when necessary, such as when an error has been made. *Erection* refers to the process of hooking and lifting members or units from the ground and attaching them to their final position in the building. As with prefabrication and preassembly, welding, attachment of pieces, and installation and tightening of bolts are done to connect the member or unit to the rest of the building, while the member or unit is still connected to the hoisting equipment, but additional holes are only drilled when necessary, such as when an error has been made. *Plumbing* refers to the process of checking and repositioning members and units to assure that they are in their correct location and proper orientation. *Permanent connection* refers to the process of completing the connections once the members and units in a certain section have been erected. This usually just involves installing and tightening any bolts that have not been installed or tightened; again, additional holes are only drilled when necessary, when an error has been made. *Decking/slab* refers to the process of placing and connecting necessary decking materials (e.g., corrugated metal decking, shear studs, rebar) which have not been installed during prefabrication or preassembly, and pouring the concrete slab.

We develop different measures for each of the critical factors. The objective of the measures is to indicate whether certain aspects of the project, such as the size of a preassembled panel, have a positive or negative impact on the critical factors. For example, the efficiency of erection may increase as the number of units to be erected is reduced; however, as the sizes of the units increase (e.g., panels with several members assembled on the ground as opposed to individual members), the erection efficiency may decrease. The measures that we identify are intended to be comprehensive and represent all the possible elements that might affect the critical factors. Although this system enables one to objectively determine which project characteristics affect the critical factors, and whether this effect is positive or negative, it does not allow the use of an ordinal scale to judge how the critical factors are affected. The measures of the critical factors are listed in Table 4-8 and are described in Appendix C.

There are several tradeoffs which must be made in choosing the project characteristics, both between the different critical factors and between the measures of a given critical factor. For the first case, adjusting conditions to optimize one critical factor may adversely affect some of the other critical factors. For example, preassembling an entire floor as one unit may greatly increase the permanent connection efficiency and worker safety, although this would likely violate the erection capacity and decrease the erection efficiency to the point of being an unacceptable option. For the second case, choosing project characteristics that increase a given critical factor in one way may result in a decrease in the critical factor for another reason. For example, erecting a few large units as opposed to many smaller units may increase erection efficiency because there are fewer lifts; however, as the size of the units to erect increases, erection becomes more difficult, possibly causing a decrease in erection efficiency. It is important that a balance be obtained when making decisions such as these; from the critical factors it is not possible to determine what the optimum choices would be, but they provide a way of identifying all the aspects that must be considered.

It is very important to note that although certain critical factor measures are associated with the design process, many of the other critical factors are affected by the aspects of the project which are chosen during the design process. Consideration of the critical factors and their measures during design may be crucial for the success of preassembled or prefabricated structures. Given current design constraints (e.g., reimbursement), it is difficult to consider all of the factors equally, but certain critical factor measures must be a priority. These critical factors and the measures listed in Table 4-8 provide a useful tool which can show designers how the other activities are affected by their decisions.

4.3 Components of Structural Systems in Steel and Precast Concrete

As mentioned in section 3.2, the use of standard structural elements, one of the critical factor measures we identify, seems to be extremely important to the ability to implement structural systems. Therefore, before we attempt to develop the new framing systems discussed in Chapter 5, we research the types of structural forms and connectors that are available, the types of structural elements and connections that are available, the systems that currently exist, and new ways of thinking about the combination of structural elements. We focus on steel and precast concrete forms, since these materials can be used for preassembly and prefabrication of structural systems.

We describe this basis for the new framing systems in detail in ATLSS Report No. 95-10, "Components of Structural Systems in Steel and Precast Concrete," [Farschman and Slaughter, 1995]. This report contains a compilation of existing standard, nonstandard, and proprietary structural forms, structural connectors, load bearing elements, and connections using steel or precast concrete. It focuses on the superstructure of buildings, including the connection of the superstructure to the foundation. We also include an analytical framework of structural elements and connections that represents all of the feasible combinations of the different types of structural elements. We also identify some examples of structural systems specifically designed with complementary elements and connections.

The compilation uses several excellent references for steel or precast concrete structural systems, aggregating their material-specific information into a general reference. The references used most extensively are *Design and Typical Details of Connections for Precast and Prestressed Concrete* (PCI, 1988), *Manual of Steel Construction--Load and Resistance Factor Design, First Edition* (AISC, 1986), *Design of Welded Structures* (Blodgett, 1966), and *Fundamentals of Building Construction* (Allen, 1990).

Sections 4.3.1 through 4.4.1 describe the major sections of "Components of Structural Systems in Steel and Precast Concrete." It should be noted, however, that the main portion of the compilation consists of many figures that illustrate examples of the concepts we discuss here; we do not include these figures here for the sake of brevity.

4.3.1 Structural Forms and Connectors

The purpose of this section in the compilation is to provide a comprehensive reference of standard, nonstandard, and proprietary steel and precast forms that currently exist, as well as the available connectors that may be used to join the forms. This is especially relevant to the design of new systems using prefabrication and preassembly, since it appears that the use of standard structural forms makes the design easier and more likely to be accepted by industry.

"Standard" structural forms and connectors are those which are easily available from steel and precast concrete manufacturers and fabricators. Standard structural forms and connectors are created in mass quantities, as opposed to "nonstandard" structural forms and connectors which require special fabrication activities, and "proprietary" structural forms and connectors which are available only through licensed manufacturers or not commercially available.

Structural "forms" are shapes of materials with known structural behaviors used alone or together as structural elements. "Connectors" are materials used to join structural forms. Section 4.3.2 describes many ways that these structural forms and connectors are used as load bearing elements.

Table 4-6: Summary of critical factors for preassembled and prefabricated structural systems.

Design Critical Factors	•Design Efficiency
Fabrication Critical Factors	•Fabrication Efficiency •Fabrication Capacity •Fabrication Safety
Prefabrication Critical Factors	•Prefabrication Efficiency •Prefabrication Capacity •Prefabrication Performance •Prefabrication Safety
Transportation Critical Factors	•Transportation Efficiency •Transportation Capacity •Transportation Performance •Transportation Safety
Handling Critical Factors	•Handling Efficiency •Handling Capacity •Handling Performance •Handling Safety
Preassembly Critical Factors	•Preassembly Efficiency •Preassembly Capacity •Preassembly Performance •Preassembly Safety
Erection Critical Factors	•Erection Efficiency •Erection Capacity •Erection Performance •Erection Safety
Plumbing Critical Factors	•Plumbing Efficiency •Plumbing Safety
Permanent Connection Critical Factors	•Permanent Connection Efficiency •Permanent Connection Safety
Decking/Slab Critical Factors	•Decking/Slab Efficiency •Decking/Slab Capacity •Decking/Slab Performance •Decking/Slab Safety

Table 4-7: **Design requirements for every project.**

Owner requirements	•Usage (e.g., office, industrial, etc.) •Location (e.g., coastal region, etc.) •Size (square footage) •Height •Special preferences
Code requirements	•Live loads for given usage •Dead loads for chosen materials •Wind loads for given location •Seismic loads for given location •Fireproofing for chosen materials
Other considerations	•Individual span length •Weight of each member •Efficiency of each member

4.3.2 Structural Elements and Connections

This section in the compilation illustrates the different ways that the structural forms and connectors described in the previous section can be used as load bearing elements to perform particular functions and how these different types of load bearing elements can be connected.

A "load bearing element" is a structural entity, such as a beam, column, wall slab, or floor slab, that must have the capacity to resist certain applied loads and is used in combination with many other load bearing elements to form a structure. A load bearing element may be composed of a single structural form or a combination of many structural forms joined with structural connectors. A "connection," as opposed to a "connector," joins two load bearing elements by using one or more types of connectors.

There are several different ways to define load bearing elements. The first is the final erected position of the element, which may be horizontal, vertical, or both (three-dimensional). A "horizontal" element, such as a beam or floor slab, is one that lies primarily within the horizontal plane, while a "vertical" element, such as a column or wall slab, lies primarily within the vertical plane. A "three-dimensional" element has both horizontal and vertical components. The second is the dimensionality of the element, which may be single, planar, or three-dimensional. A "single" element, such as a beam or column, is an element that can be approximated as extending in one direction, as opposed to a "planar" element, such as a wall or floor slab, which extends in two directions, and a "three-dimensional" element which extends in three directions. The third way to define load bearing elements is by the continuity of the element, whether it is continuous or discontinuous. A "continuous" element has a uniform, unbroken surface, while the surface of a "discontinuous" element is skeletal and interrupted. The final way to define load bearing elements is by the type of material that the element is made of, which in this context may be steel or precast concrete. Each of these factors is used to classify the different types of elements in "Components of Structural Systems in Steel and Precast Concrete."

4.3.3 Analytical Framework of Structural Elements and Connections

This section in "Components of Structural Systems in Steel and Precast Concrete" explores all of the technically feasible combinations of elements, by orientation and assembly continuity, revealing the similarities and differences in the way elements may be combined. It provides a framework for the analysis of structural systems, specifically the interaction between the orientation, dimensionality, and continuity of the structural forms and the nature of the connections. This portion of the report goes into more detail as to the possible orientations of the elements; however, connection details are not shown and the elements are not classified by material but treated as having generic properties. This section is relevant to preassembly and prefabrication by establishing new ways of thinking about this issue and possibly leading to ideas for new structural systems.

4.3.4 Examples of Structural Systems

With the intention of providing more efficient methods of construction than the traditional method of stick-building members, new structural systems have been developed. For our purposes, a "structural system" refers to a set of structural elements specially designed to fit together using specific connections to simplify erection. Structural systems are designed for certain conditions or loads and they are most efficient when the elements and connections are used together.

In this section in the compilation, examples of structural systems specifically designed with complementary elements and connections are represented. These examples provide insight into different approaches that may be taken in response to the need for new systems for efficient prefabrication and preassembly.

Table 4-8: Measures of critical factors.

	Efficiency	Capacity	Performance	Safety
Design	•Standardization of structural elements •Standardization of structural configurations •Repetition of design units •Member redundancy			
Fabrication	•Connection complexity •Number of connections per member •Member complexity •Number of members •Tolerances •Resources •Special equipment requirements •Special activity requirements	•Equipment capacity •Shop layout •Degree of automation		•Labor hours •Danger exposure •Special personal equipment
Prefabrication	•Connection complexity •Number of connections per member •Size of members •Weight of members •Member complexity •Ease of member placement •Measurement (w.r.t. other members) •Number of members per unit •Number of units •Tolerances •Resources •Special equipment requirements •Special activity requirements •Special site constraints	•Equipment capacity •Laydown area (space)	•Stability •Durability •Capacity	•Labor hours •Danger exposure •Special personal equipment
Transportation	•Number of units •Size of units •Weight of units •Density of units •Ease of stacking •Resources [continued on next page]	•Equipment capacity	•Stability •Durability •Capacity	•Labor hours •Danger exposure •Special personal equipment

Table 4-8 (cont): Measures of critical factors.

	Efficiency	Capacity	Performance	Safety
Transportation (cont)	[continued from previous page] •Special equipment requirements •Special activity requirements •Special site constraints	[continued from previous page]	[continued from previous page]	[continued from previous page]
Handling	•Number of units •Size of units •Weight of units •Ease of stacking •Resources •Special equipment requirements •Special activity requirements •Special site constraints	•Equipment capacity •Laydown area (space)	•Stability •Durability •Capacity	•Labor hours •Danger exposure •Special personal equipment
Preassembly	•Connection complexity •Number of connections per member •Size of members •Weight of members •Member complexity •Ease of member placement •Measurement (w.r.t. other members) •Number of members per unit •Number of units •Tolerances •Resources •Special equipment requirements •Special activity requirements •Special site constraints	•Equipment capacity •Laydown area (space)	•Stability •Durability •Capacity	•Labor hours •Danger exposure •Special personal equipment

Table 4-8 (cont): Measures of critical factors.

	Efficiency	Capacity	Performance	Safety
Erection	•Connection complexity •Number of connections per unit •Size of units •Weight of units •Unit complexity •Ease of unit placement •Number of units •Tolerances •Height •Resources •Special equipment requirements •Special activity requirements •Special site constraints	•Equipment capacity	•Stability •Durability •Capacity	•Labor hours •Danger exposure •Special personal equipment
Plumbing	•Number of units to plumb •Ease of plumbing •Tolerances •Resources •Special equipment requirements •Special activity requirements			•Labor hours •Danger exposure •Special personal equipment
Permanent Connection	•Connection complexity •Number of connections per unit •Unit complexity •Number of units •Height •Resources •Special equipment requirements •Special activity requirements			•Labor hours •Danger exposure •Special personal equipment
Decking/Slab	•Floor complexity •Number of floors •Congestion of floor •Resources •Special equipment requirements •Special activity requirements •Special site constraints	•Equipment capacity	•Stability •Capacity	•Labor hours •Danger exposure •Special personal equipment

Chapter 5: New Preassembled Structural Systems

5.1 General Introduction

In this chapter, we describe the concepts for three new systems that we develop in the course of this research to take advantage of the inherent benefits of preassembly and prefabrication. We develop these systems in response to the general conclusions drawn from the site observations, discussed in section 4.1.3, and the critical factors of structural systems that we discussed in section 4.2. We also modify these systems according to feedback from industry members, through site interviews, which we discussed in section 3.3. Table 5-1 lists the possible variations of the three new systems.

These system concepts can be seen as a "proof test" of the approach and specifically the use of the critical factors and the framework for structural systems. These concepts are an example of the application of the methodology. In addition, it is hoped that they provide a basis for structural designers, fabricators, and erectors to experiment in the preassembly of structural systems.

Each of these systems is intended for use in low to mid-rise buildings, including commercial, light industrial, and residential uses, primarily in areas with low seismic activity and low wind load. Additional attachments to the columns and panels can extend applications to take into account higher loads, although this might reduce the benefits that the systems offer.

From the literature, it is expected that the potential advantages these systems offer include reduced construction duration, labor requirements, and direct and indirect costs for structural erection, and increased safety, complying with all regulations associated with erection from the Occupational Safety and Health Administration (OSHA).

The specific concepts also incorporate additional advantages, including the creation of a continuous rigid system with smaller members and a shallow floor using composite action between the cast-in-place concrete floor slab and the steel structural elements, and the use of existing structural members combined in a novel arrangement.

We discuss the three structural systems in sections 5.2 to 5.4. For each system, we describe our objectives in developing the system, and we give a general description of the system, including the possible configurations and panel options. Then we describe the results of a series of analyses of the systems. First, we analyze each system with respect to the critical factors of structural systems discussed in section 4.2, to determine whether the systems were effectively designed for preassembly. We use this analysis to identify the most promising variations of the new systems and those with inherent flaws that would cause major problems during construction. After choosing the best options with the critical factors analysis, we perform a structural analysis on the promising configuration and panel options of each system. For a prototype building using each of the new systems, we perform a basic structural analysis to determine whether the buildings have the capacity to withstand basic gravity loads. In addition to checking the structural capacity and stability of the systems, the structural analyses also provide the approximate sizes and numbers of members that would work for each system, as well as the approximate connection types. We use this information in the project analyses, to calculate the approximate duration for construction of each building, as well as worker air time. From this information, we compare the systems to determine which are most advantageous in the areas of duration and worker safety. We discuss this with the conclusions in section 5.5, along with explicit tradeoffs that we identify during the course of performing these analyses.

Table 5-1: Summary of new systems.

System	Configuration Options	Panel Options
System 1	1) Box-beam girder configuration, plate alignment mechanism with column rods, one-story columns connected with splice sleeves 2) I-beam girder configuration, plate alignment mechanism with column rods, one-story columns connected with splice sleeves 3) Box-beam girder configuration, preattached angles act as alignment mechanism and form a I-shape configuration inside one-story columns 4) I-beam girder configuration, preattached angles act as alignment mechanism and form a cruciform configuration inside one-story columns 5) Box-beam girder configuration, ATLSS Connector acts as alignment mechanism, one or multi-story columns 6) Box-beam girder configuration, standard bolted connection, one or multi-story columns 7) I-beam girder configuration, standard bolted connection, one or multi-story columns	1) Open-bar joist spanning elements 2) Standard W-shape beam spanning elements 3) Castellated beam spanning elements 4) "Modified" castellated beam spanning elements
System 2	I-beam girder configuration, multi-story steel columns with rigid beam stubs in transverse direction, standard splice connection	1) Open-bar joist spanning elements 2) Standard W-shape beam spanning elements 3) Castellated beam spanning elements 4) "Modified" castellated beam spanning elements
System 3	1) Multi-story steel or precast columns with steel collars 2) One story precast concrete "lily" columns	Precast concrete panel

5.2 System 1: "Tubular Column System"

5.2.1 Objectives and General Description of System 1

The "Tubular Column System" is a complete structural framing system which consists of tubular vertical load-bearing elements, horizontal load-bearing elements, and floor surfaces. It incorporates special connections to enable erection with little manual alignment or attachment.

In developing this system, our objectives were to incorporate the decking as part of the preassembled panel, to use repetitive bay-size panels, while minimizing member redundancy, to provide a possible alignment mechanism for the panels, and to use standard structural elements.

The vertical elements in System 1 are concrete-filled steel tubular columns. The columns may be one-story or multi-story depending on the connection option chosen. The different possibilities for the column to column connection are described in section 5.2.1.1.

The horizontal elements are preassembled panels for spanning among four columns (one bay). The members supporting the spanning elements of the panels are channels, some of which have plates attached to their bottom flange that will be attached to the channel of an adjacent panel to connect the channels and provide extra strength and stability to the girder. Depending on the configuration option chosen, the channels are either placed with the open face on the exterior edge of the panel so that when two panels are joined the attached channels form a box-beam configuration (Figure 5-1) or with the web of the channel on the exterior edge of the panel so the two channels from adjacent panels form an I-beam configuration (Figure 5-2). This is a one-way system; there are no girders spanning from column to column in the direction parallel to the spanning elements. The different possibilities for the panel to column connection are described in section 5.2.1.1 (see Table 5-1).

There are four options for the type of spanning elements in the preassembled panels, described in section 5.2.1.2 (see Table 5-1). The preassembled panel also includes corrugated metal decking and shear studs (with the exception of innovative panel option 4). A cast-in-place concrete floor slab produces composite action with the steel elements.

5.2.1.1 Configuration Options for System 1

Configuration Option 1: (Box-beam girder configuration, plate alignment mechanism with column rods, one-story columns connected with splice sleeves)

In configuration option 1, each one-story, concrete-filled steel tubular column has steel rods protruding from the concrete at the top of the tube and receiving sleeves at the bottom of the tube to provide continuous column to column connections when the columns are in place (the column-column connection could also include splices if the expected load requires it). Seating angles are preattached to the top of each column and smaller top angles at the bottom of each column to provide a means for panel to column connections (Figure 5-3).

In the panels, the channels are placed with the open face on the exterior edge of the panel so that when two panels are joined the attached channels form a box-beam configuration (Figures 5-1 and 5-4). To join the adjacent channels, and to increase the torsional resistance of the channels, a bottom plate is shop-bolted to one of the channels and field bolted to the other channel (using "break-off" bolts which do not require access from the other side).

Another plate attached to each channel at the corners of each panel acts as an alignment mechanism, as well as part of the panel to column connection (Figure 5-4). Each plate has an oversized hole in it which fits over a steel rod from the column below. The panel also rests on the seating angles which are preattached to the lower column. Once the four panels are set on the lower tubular column, the column for the next story is inserted over the four rods, and its bottom surface rests on the plates from the four panels (Figure 5-5). The top angles preattached to the upper column are attached to the channels, and the receiving sleeves in the upper column are grouted. When the upper columns of one story are erected, the topping slab for that floor can be poured.

Configuration Option 2: (I-beam girder configuration, plate alignment mechanism with column rods, one-story columns connected with splice sleeves)

In configuration option 2, like configuration option 1, each one-story, concrete-filled steel tubular column has steel rods protruding from the concrete at the top of the tube and receiving sleeves at the bottom of the tube to provide continuous column to column connections when the columns are in place (the column-column connection could also include splices if the expected load requires it). Seating angles are preattached to the top of each column and smaller top angles at the bottom of each column to provide a means for panel to column connections (Figure 5-3).

In the panels, the channels are placed with the webs on the exterior edge of the panel so that when two panels are joined the attached channels form an I-beam with a double web (Figure 5-2 and 5-6). The channels of adjacent panels are bolted together at intervals along their length between the columns to increase torsional resistance. The channels may also be connected with a bottom plate, as in configuration option 1, if this is required to provide sufficient strength and stability of the girders. (For the loads in our example application, discussed in section 5.2.3, we find that this bottom plate is not necessary.)

A plate attached to each channel at each corner of the panel acts as an alignment mechanism, as well as part of the panel to column connection (Figure 5-6). Each plate has an oversized hole in it which fits over a steel rod from the column below. The panel also rests on the seating angles which are preattached to the lower column. Once the four panels are set on the lower tubular column, the column for the next story is inserted over the four rods, and its bottom surface rests on the plates from the four panels (Figure 5-7). The top angles preattached to the upper column are attached to the channels, and the receiving sleeves in the upper column are grouted. When the upper columns of one story are erected, the topping slab for that floor can be poured.

Configuration Option 3: (Box-beam girder configuration, preattached angles act as alignment mechanism and form a I-shape configuration inside one-story columns)

In configuration option 3, the one-story steel tubular columns prefilled with concrete have metal plates (with shear studs attached) within the tube on the top and bottom of the concrete to space the concrete from the floor members and to provide a bearing surface for the connection (Figure 5-8). Seating angles are preattached to the top of each column and smaller top angles at the bottom of each column to provide a means for panel to column connections.

In the panels, the channels are placed with the open face on the exterior edge of the panel so that when two panels are joined the attached channels form a box-beam configuration (Figures 5-1 and 5-9). To join the adjacent channels, and to increase the torsional resistance of the channels, a bottom plate is shop-bolted to one of the channels and field bolted to the other channel (using "break-off" bolts which do not require access from the other side).

At each corner of the preassembled panels, an angle is attached that extends below the bottom of the panel and above the top of the panel (Figure 5-9). This angle acts as the alignment mechanism for erection and as the bearing element for the permanent connection. The angle fits within the tubular column below, rests upon the plate within the column on top of the precast concrete, and is attached to the column. The panel also rests on the seating angles which are preattached to the lower column. Once the four panel angles (which have a rectangular configuration) are set within the tubular column, the column for the next story is inserted over the four angles (Figure 5-10). The bottom surface of the panel rests upon the top of lower column, and the upper column rests upon the tops of the panel angles on the plate within the column, spaced up from the floor the height of the cast-in-place slab. The top angles preattached to the upper column are attached to the channels. After the upper columns of one story are erected, the topping slab for that floor can be poured. When the slab is poured, the remaining column volume from the top of the plate to the bottom of the slab for the lower column, and from the top of the slab to the bottom of the plate for the upper column, is grouted.

Configuration Option 4: (I-beam girder configuration, preattached angles act as alignment mechanism and form a cruciform configuration inside one-story columns)

Configuration option 4 is similar to configuration option 3 with one-story steel tubular columns prefilled with concrete and metal plates (with shear studs attached) within the tube on the top and bottom of the concrete to space the concrete from the floor members and to provide a bearing surface for the connection (Figure 5-8). Seating angles are preattached to the top of each column and smaller top angles at the bottom of each column to provide a means for panel to column connections.

In the panels, the channels are placed with the webs on the exterior edge of the panel so that when two panels are joined the attached channels form an I-beam with a double web (Figures 5-2 and 5-11). The channels of adjacent panels are bolted together at intervals along their length between the columns to increase torsional resistance. The channels may also be connected with a bottom plate, as in configuration options 1 and 3, if this is required to provide sufficient strength and stability of the girders. (For the loads in our example application, discussed in section 5.2.3, we find that this bottom plate is not necessary.)

At each corner of the preassembled panels, an angle is attached that extends below the bottom of the panel and above the top of the panel. This angle acts as the alignment mechanism for erection and as the bearing element for the permanent connection (Figure 5-11). The angle fits within the tubular column below, and rests upon the plate within the column on top of the precast concrete. The panel also rests on the seating angles which are preattached to the lower column. Once the four panel angles (which have a cruciform configuration) are set within the tubular column, the column for the next story is inserted over the four angles (Figure 5-12). The bottom surface of the panel rests upon the top of the lower column, and the upper column rests upon the tops of the panel angles on the plate within the column, spaced up from the floor the height of the cast-in-place slab. The top angles preattached to the upper column are attached to the channels. After the upper columns of one story are erected, the topping slab for that floor can be poured. When the slab is poured, the remaining column volume from the top of the plate to the bottom of the slab for the lower column, and from the top of the slab to the bottom of the plate for the upper column, is grouted.

Configuration Option 5: (Box-beam girder configuration, ATLSS Connector acts as alignment mechanism, one or multi-story columns)

In configuration option 5, one-story or multi-story concrete-filled steel tubular columns may be used, with any appropriate method of joining the columns. Each column has the receiving portion of an ATLSS connector at each panel location to provide a means for panel to column connections.

In the panels, which have four ATLSS Connectors near the bottom part of the channel webs, the channels are placed with the open face on the exterior edge of the panel so that when two panels are joined the attached channels form a box-beam configuration (Figure 5-1). To join the adjacent channels, and to increase the torsional resistance of the channels, a bottom plate is shop-bolted to one of the channels and field bolted to the other channel (using "break-off" bolts which do not require access from the other side).

The ATLSS Connectors provide an alignment mechanism for the panels, as well as forming part of the panel to column connection. The panels slip into the four receiving ATLSS Connectors which are preattached to the lower columns, and the angles preattached to the top of the channels are connected to the columns. This connection can be made with special bolts which must be embedded in the concrete, or the concrete may be poured after erection and "blind" bolts may be used to connect the angles to the columns.

Configuration Option 6: (Box-beam girder configuration, standard bolted connection, one or multi-story columns)

In configuration option 6, one-story or multi-story concrete-filled steel tubular columns may be used, with any appropriate method of joining the columns. Each column has seating angles attached at each panel location to provide a means for panel to column connections.

In the panels, which have a top angle preattached to the top of the channels for the panel to column connections, the channels are placed with the open face on the exterior edge of the panel so that when two panels are joined the attached channels form a box-beam configuration (Figure 5-1). To join the adjacent channels, and to increase the torsional resistance of the channels, a bottom plate is shop-bolted to one of the channels and field bolted to the other channel (using "break-off" bolts which do not require access from the other side).

This configuration does not provide the alignment mechanism included in the other configuration options. The panels rest on the seating angles which are preattached to the lower columns, and the angles preattached to the top of the channels are connected to the columns. This connection can be made with special bolts which must be embedded in the concrete, or the concrete may be poured after erection and "blind" bolts may be used to connect the angles to the columns.

Configuration Option 7: (I-beam girder configuration, standard bolted connection, one or multi-story columns)

In configuration option 7, like configuration option 6, one-story or multi-story concrete-filled steel tubular columns may be used, with any appropriate method of joining the columns. Each column has seating angles attached at each panel location to provide a means for panel to column connections.

In the panels, which have a top angle preattached to the top of the channels for the panel to column connections, the channels are placed with the webs on the exterior edge of the panel so that when two panels are joined the attached channels form an I-beam configuration with a double web (Figure 5-2). The channels of adjacent panels are bolted together at intervals along their length between the columns to increase torsional resistance. The channels may also be connected with a bottom plate, as in configuration option 6, if this is required to provide sufficient strength and stability of the girders.

Like option 6, this configuration does not provide the alignment mechanism included in the other configuration options. The panels rest on the seating angles which are preattached to the lower columns, and the angles preattached to the top of the channels are connected to the columns. This connection can be made with special bolts which must be embedded in the concrete, or the concrete may be poured after erection and "blind" bolts may be used to connect the angles to the columns.

5.2.1.2 Panel Options for System 1

Panel Option 1: (Open-Bar Joist Spanning Elements)

In panel option 1, open-bar joists span from one channel to the opposite parallel channel. Corrugated metal decking covers the complete surface of the panel, and shear studs are placed along the tops of the channels and along the joist line. For the box-beam girder configuration, the bar joists are connected to the channel webs with angles that are preattached to the channels (Figure 5-13), and for the I-beam girder configuration, the bar joists are connected similarly, although they are slightly lower than the top of the channel (Figure 5-14). The open-bar joists have to be designed and fabricated to specific span lengths.

Panel Option 2: (Standard W-Shape Beam Spanning Elements)

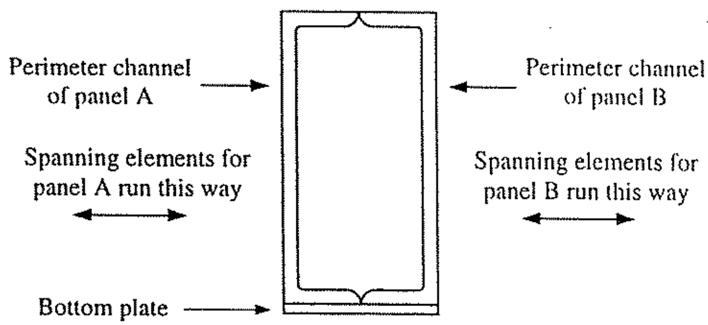
In panel option 2, standard W-shape beams span from one channel to the opposite parallel channel. Corrugated metal decking covers the complete surface of the panel, and shear studs are placed along the tops of the channels and along the beam line. For the box-beam girder configuration, the beams are connected to the channel web with a shear angle (Figure 5-15), and for the I-beam girder configuration, the beams are connected similarly, but must be coped (Figure 5-16). The W-shape beams do not require special design or fabrication requirements for specific lengths but can be cut to length from common stock.

Panel Option 3: (Castellated Beam Spanning Elements)

In panel option 3, castellated beams span from one channel to the opposite parallel channel. Corrugated metal decking covers the complete surface of the panel, and shear studs are placed along the tops of the channels and along the joist line. Like panel option 2, for the box-beam girder configuration, the castellated beams are connected to the channel web with a shear angle (Figure 5-17), and for the I-beam girder configuration, the castellated beams are connected similarly, but must be coped (Figure 5-18). The castellated beams do not require special design or fabrication requirements for specific lengths but can be cut to length from common stock.

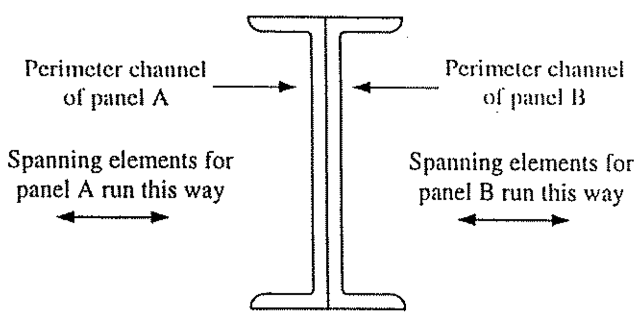
Panel Option 4: ("Modified" Castellated Beam Spanning Elements)

In panel option 4, castellated beams with an additional half of the castellation welded along the top flange span from one channel to the opposite parallel channel. Corrugated metal decking is placed between the top flanges of the castellated beams so that the additional half-castellation protrudes above the decking to act as a shear element for a composite deck, replacing the need for shear studs along the spanning elements (Figure 5-19). This panel option is an innovative concept which has never been attempted and must be tested thoroughly before it can actually be put into practice. (However, a representative of Chaparral Steel Company, a major manufacturer of castellated beams, believes this to be an interesting and viable option.) For the box-beam girder configuration, the castellated beams are connected to the channel web with a shear angle (Figure 5-20), and for the I-beam girder configuration, the castellated beams are connected similarly, but must be coped (Figure 5-21). The castellated beams do not require special design or fabrication requirements for specific lengths but can be cut to length from common stock.



(Spanning elements, metal decking, and concrete slab not shown.)

Figure 5-1: Channel girders with box-beam configuration (configuration options 1, 3, 5, and 6).



(Spanning elements, metal decking, and concrete slab not shown.)

Figure 5-2: Channel girders with I-beam configuration (configuration options 2, 4, and 7).

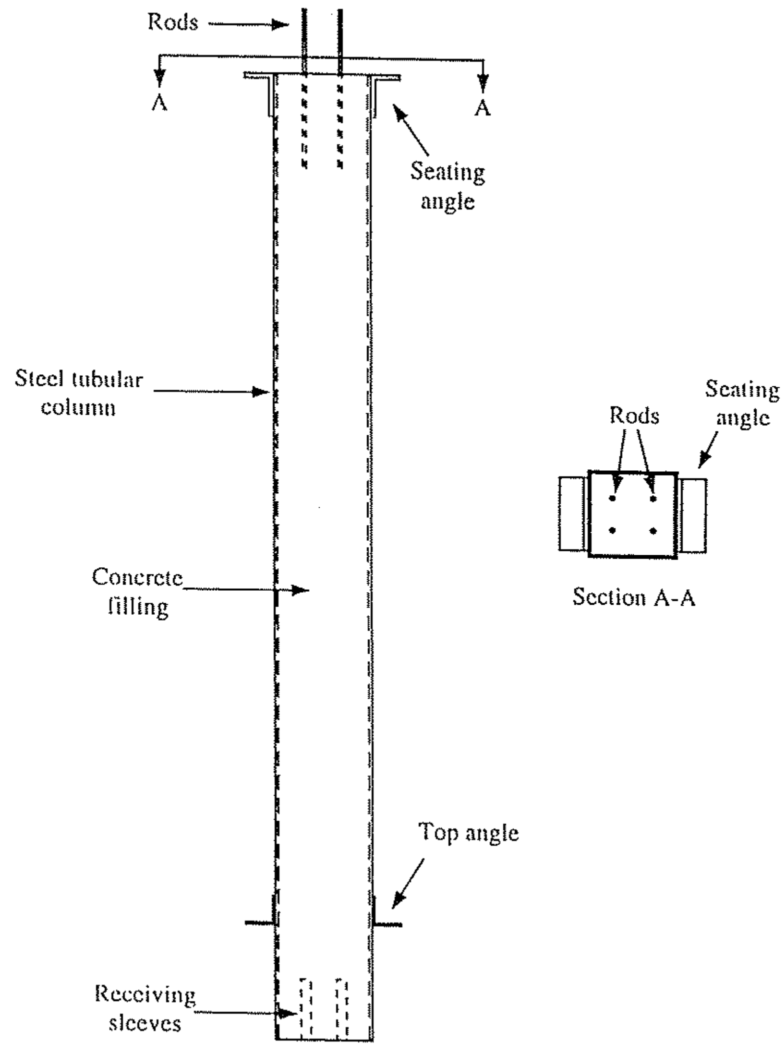
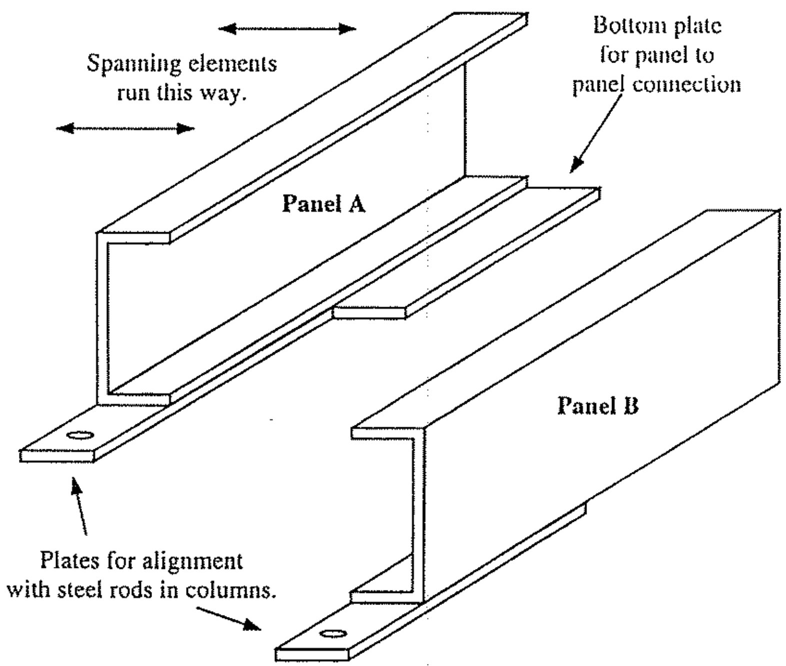


Figure 5-3: Column used in configuration options 1 and 2.



(Spanning elements and metal decking not shown.)

Figure 5-4: Channel girders and alignment mechanism used in configuration option 1.

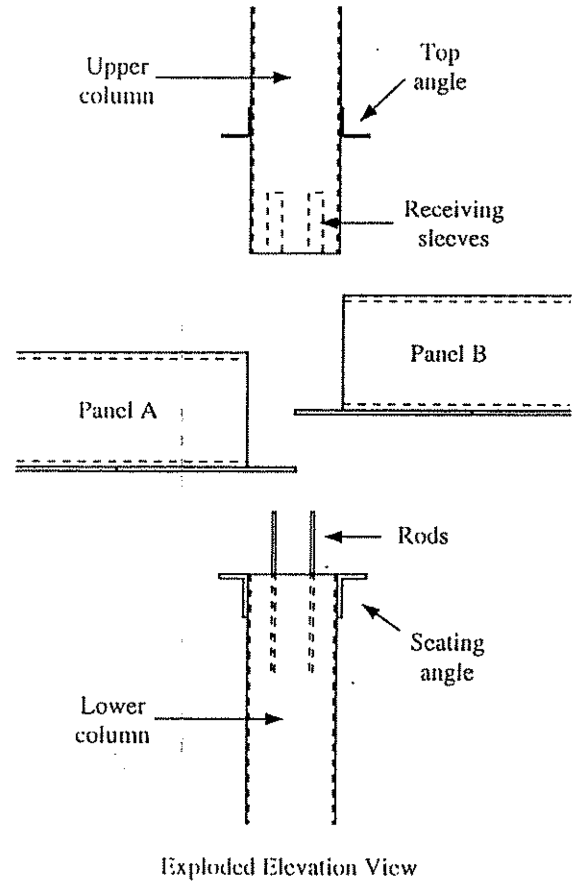
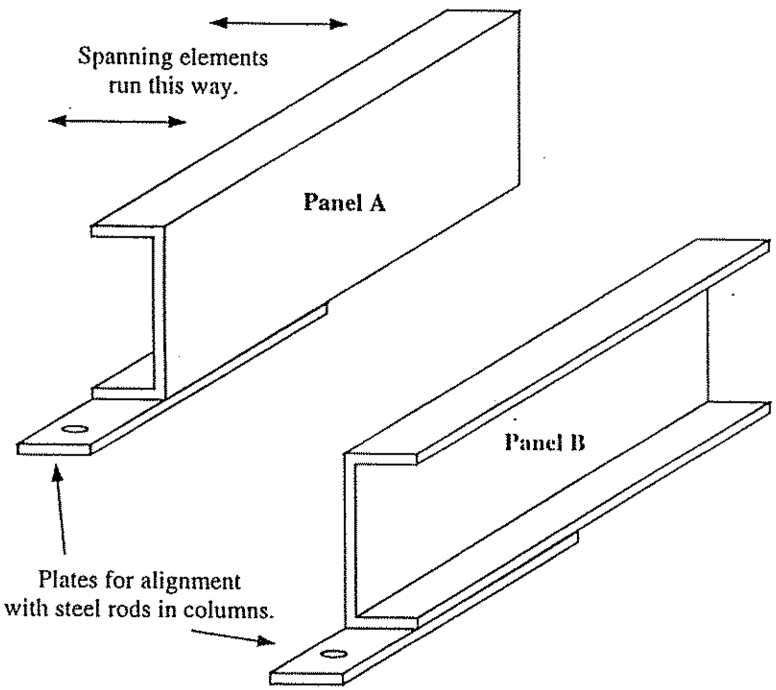


Figure 5-5: Overall system view of configuration option 1.



(Spanning elements and metal decking not shown.)

Figure 5-6: Channel girders and alignment mechanism used in configuration option 2.

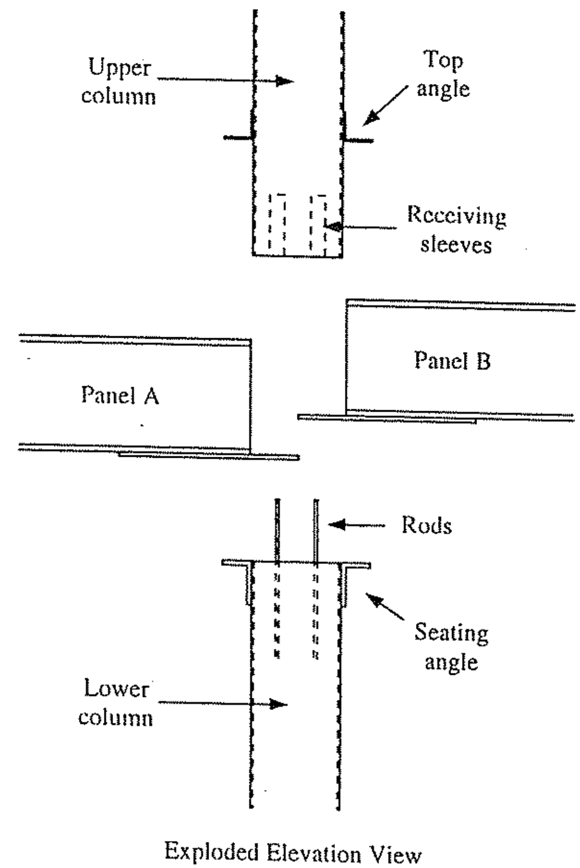


Figure 5-7: Overall system view of configuration option 2.

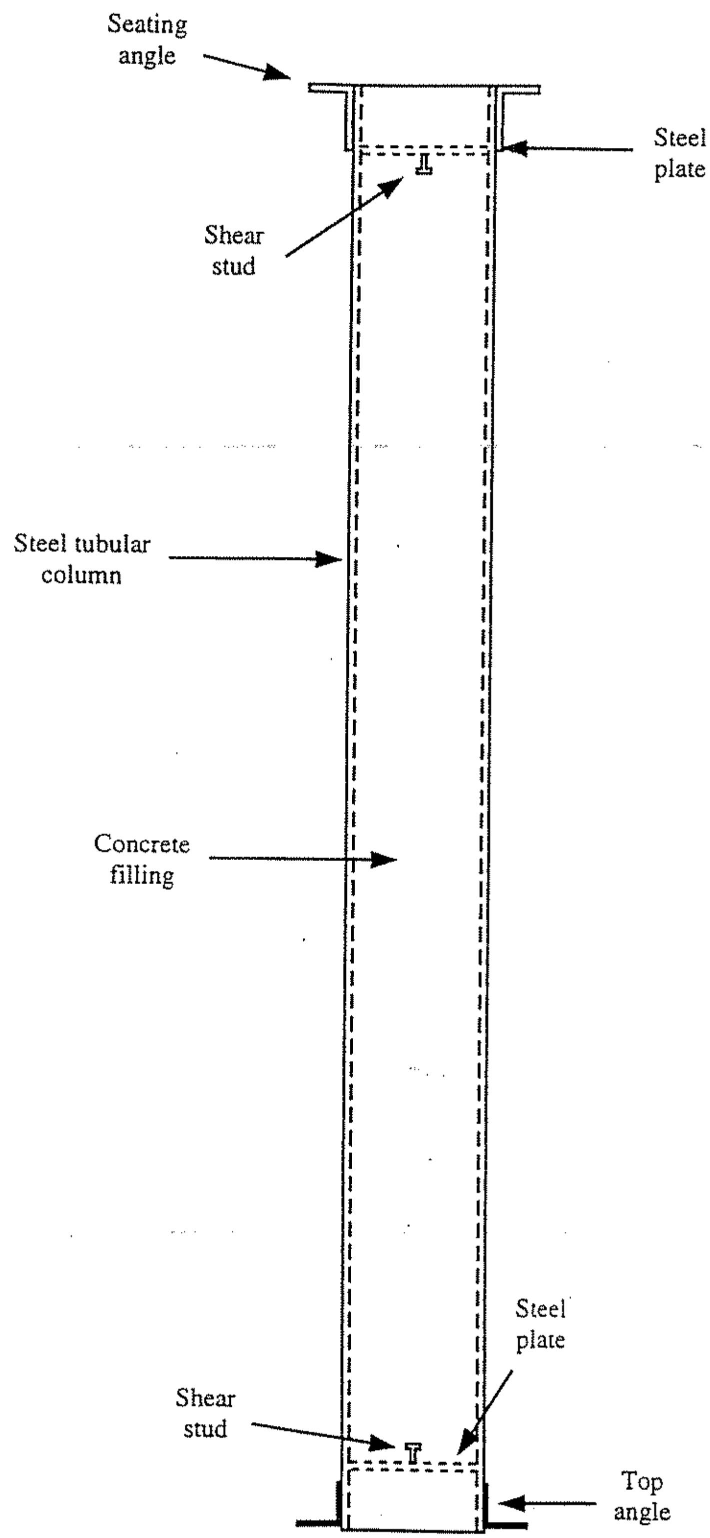
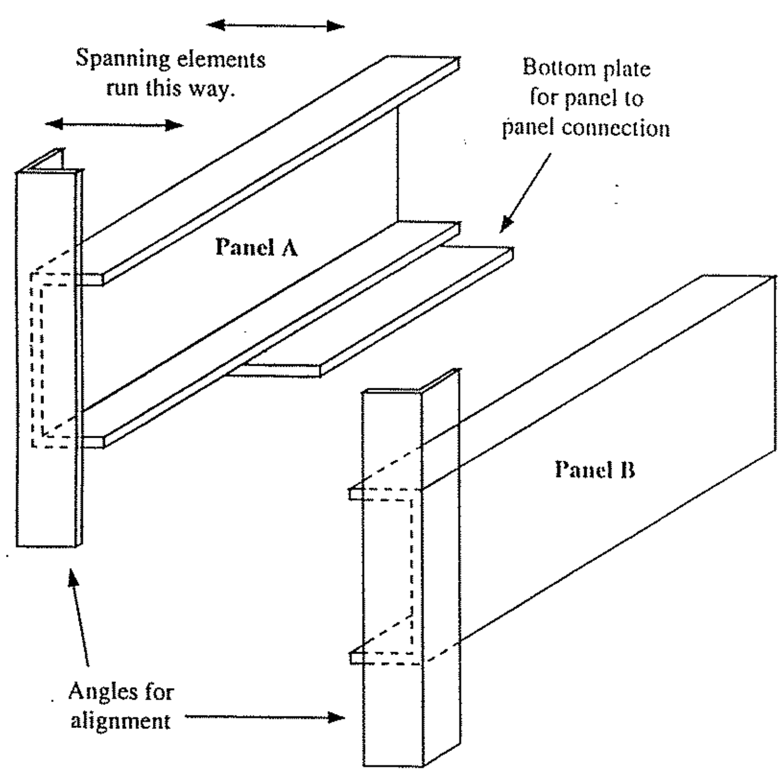
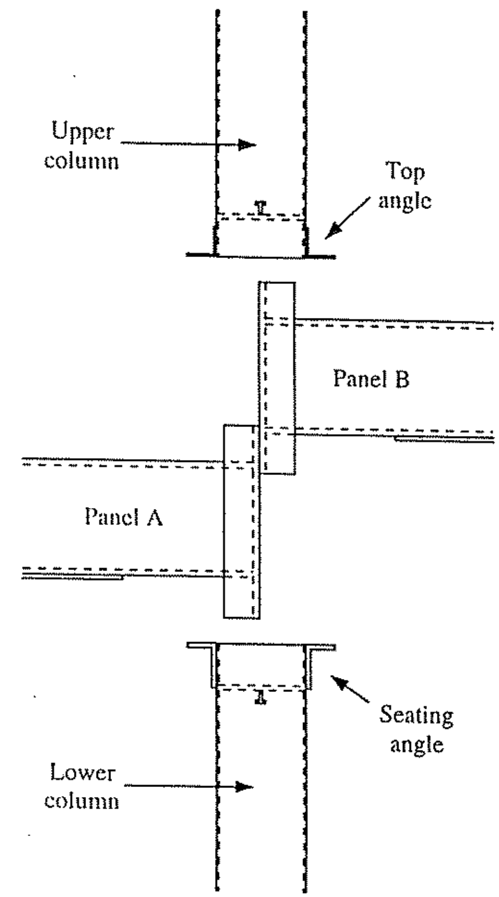


Figure 5-8: Column used in configuration options 3 and 4.



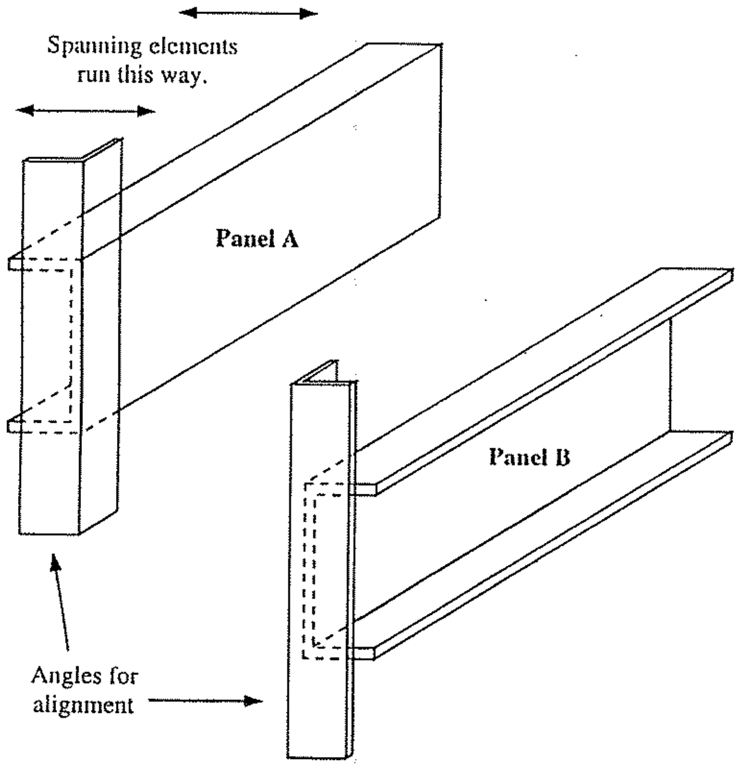
(Spanning elements and metal decking not shown.)

Figure 5-9: Channel girders and alignment mechanism used in configuration option 3.



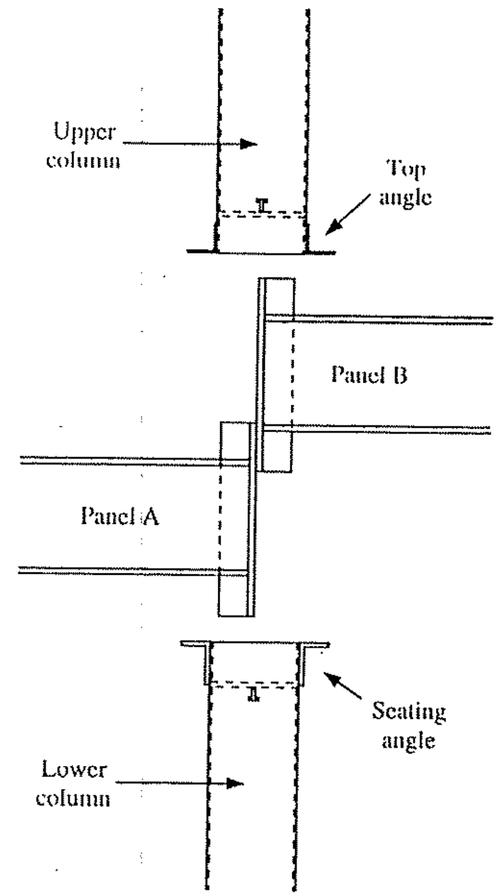
Exploded Elevation View

Figure 5-10: Overall system view of configuration option 3.



(Spanning elements and metal decking not shown.)

Figure 5-11: Channel girders and alignment mechanism used in configuration option 4.



Exploded Elevation View

Figure 5-12: Overall system view of configuration option 4.

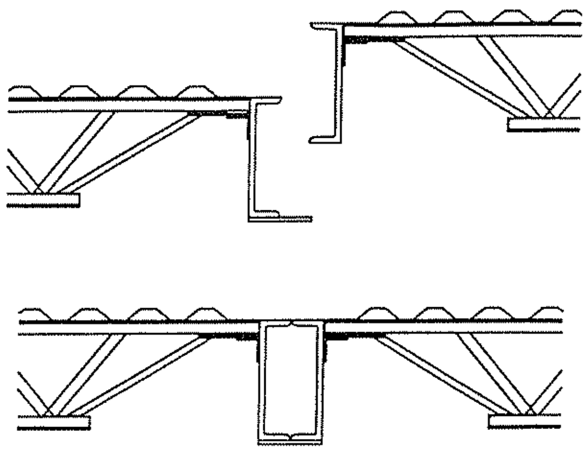


Figure 5-13: Panel option 1, with box-beam girder configurations.

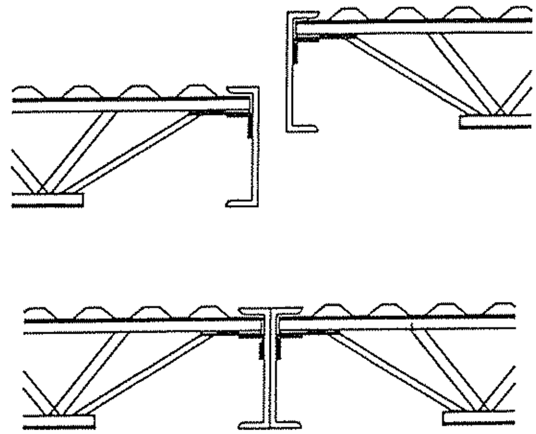


Figure 5-14: Panel option 1, with I-beam girder configurations.

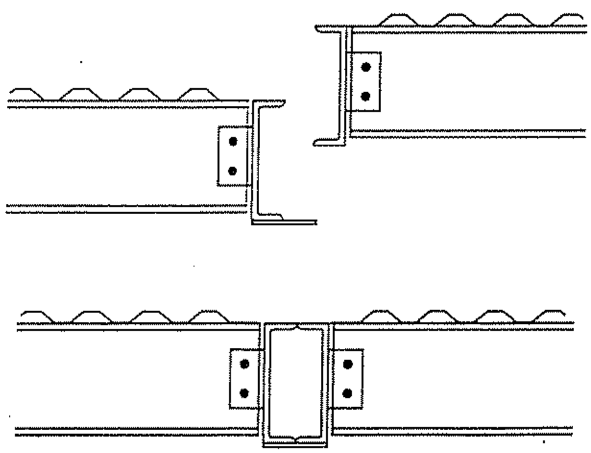


Figure 5-15: Panel option 2, with box-beam girder configurations.

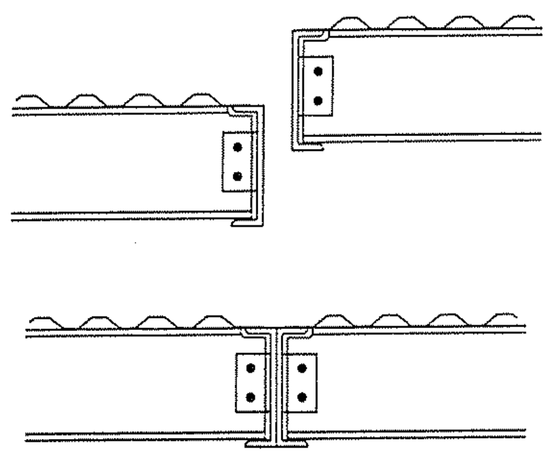


Figure 5-16: Panel option 2, with I-beam girder configurations.

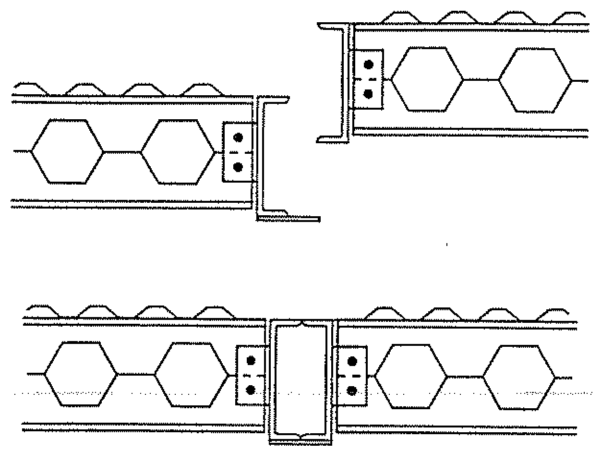


Figure 5-17: Panel option 3, with
box-beam girder configurations.

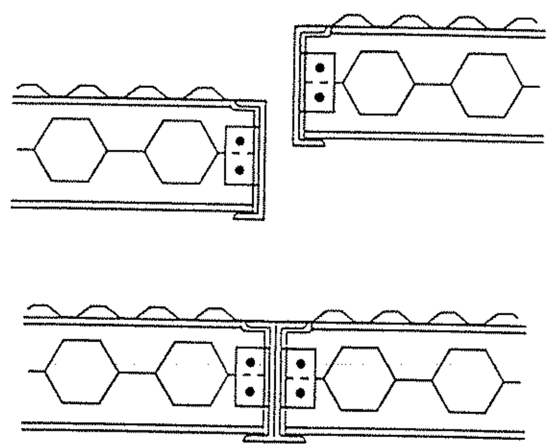


Figure 5-18: Panel option 3, with
I-beam girder configurations.

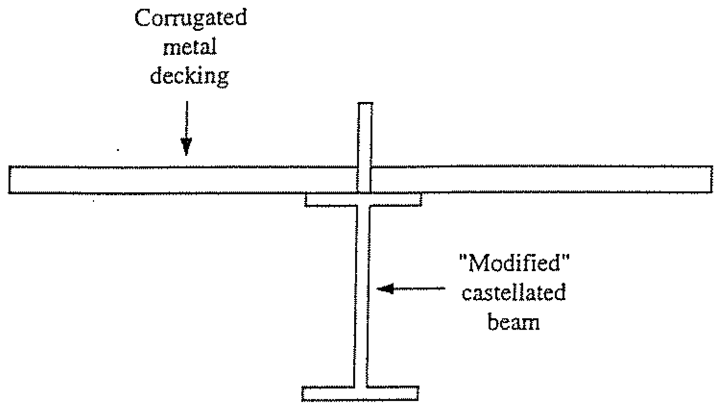


Figure 5-19: Cross-sectional view of panel option 4.

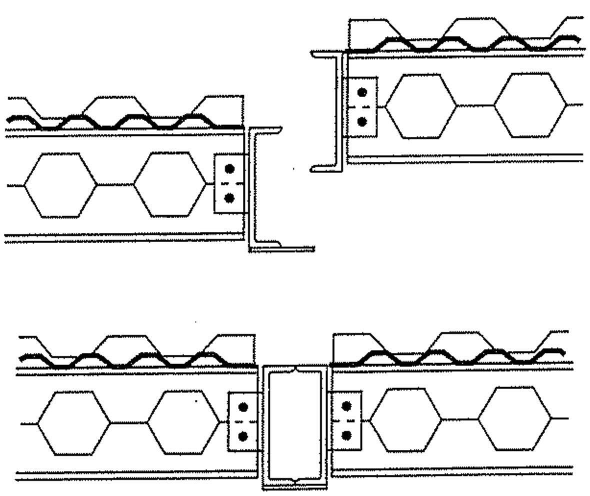


Figure 5-20: Panel option 4, with box-beam girder configurations.

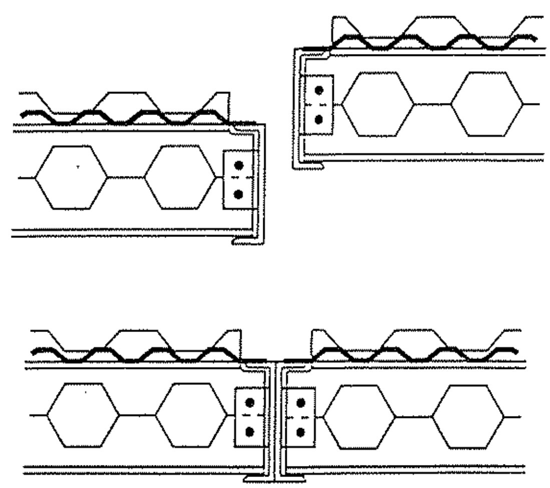


Figure 5-21: Panel option 4, with I-beam girder configurations.

5.2.2 Analysis of System 1 with Respect to Critical Factors

As discussed in section 3.4.1, before we perform a structural performance analysis or project impact analysis on any of the variations of System 1, we perform an analysis of each variation with respect to the critical factors to determine whether the systems are effectively designed for preassembly. We use this analysis to identify the most promising the variations of the new system, as well as any inherent flaws in any of the variations that may cause major problems during construction. This type of analysis also allows us to identify tradeoffs between the different types of construction.

In this section, we describe the results of these analyses for each configuration, noting the advantages and disadvantages of the new systems when compared to traditionally constructed systems. In performing this analysis, we are comparing the activities that we assume would be involved in constructing the new systems to the activities associated with traditional construction. When making this comparison, we notice that there are some activities involved in the construction of the new systems that are not included in traditionally constructed systems (e.g., preassembly); however, there are also activities that as a result of using the new system are more efficient or safer than if the system were traditionally constructed (e.g., erection). These are the types of tradeoffs that this analysis brings out.

Table 5-2 shows the results of the critical factor analysis for configuration option 1, the potential advantages and disadvantages of this system. The results are similar for the remaining configuration options, so in Tables 5-3 through 5-8, we list how the other configuration options differ from configuration option 1, whether the difference is an advantage or disadvantage, rather than repeating similar results. It should be noted that for all of the variations of System 1, redundant members are used (i.e., two channels rather than one W-shape beam); however, they are combined in such a way as to take advantage of their redundancy (two smaller members combine to provide the strength of one larger member). This may or may not be a disadvantage, but it not listed in Table 5-2. Other advantages and disadvantages not incorporated in Table 5-2 include the additional advantage of eliminating fireproofing requirements, provided by the concrete filled tubes and the additional possible disadvantage of needing a larger crane to lift the heavy panels than would be required for traditional construction.

From the critical factor analysis of the different variations of System 1, configuration options 1 and 2 appear to be the variations that are most promising. Therefore, we only consider these two options in the structural analysis, which we describe in the next section.

Table 5-2: **Results of critical factor analysis for configuration option 1, System 1.**

Potential Advantages	Potential Disadvantages
•Transportation and handling efficiencies may slightly increase, due to the use of one-story rather than multi-story columns. •Erection efficiency may increase, due to (1) the alignment mechanism provided by the plate and (2) the need to erect significantly fewer members, when compared to traditional construction. •Worker safety during erection may significantly increase, due to fewer labor hours, especially in the air, and lower danger exposure. •Permanent connection efficiency may increase, due to the transfer of much of the permanent connection stage to the ground during preassembly, instead of in the air after erection. •Worker safety during permanent connection may significantly increase, due to the transfer of much of the permanent connection stage to the ground during preassembly, instead of in the air after erection. •Decking efficiency may increase, due to the transfer of many of the decking activities (decking installation, shear studs) to the ground during preassembly, instead of in the air after erection. •Worker safety during decking may significantly increase, due to the transfer of many of the decking activities (decking installation, shear studs) to the ground during preassembly.	•Design efficiency may decrease, due to the nonstandard structural configuration, which may require special design consideration. •Fabrication efficiency may decrease, due to (1) the necessary hole drilling and bolt installation (or welding) and complicated topology associated with the plate attached to the bottom of the channels, (2) the increase in the number of members (due to the channels and one-story columns) when compared to traditional construction, and (3) the need for strict tolerances, especially associated with the aligning plate. •Transportation and handling efficiencies may slightly decrease, due to the heavy concrete-filled tubes and the increased number of members. •Preassembly efficiency may decrease, due to the need for strict tolerances, associated with the alignment of the panel during erection. •Erection efficiency may decrease, due to the larger and heavier units to be erected, when compared to traditional construction, as well as the fact that there are more connections per unit than stick-built members, and it is more difficult to balance the larger units. •Erection efficiency may decrease if there are alignment difficulties. •Erection performance, or stability and ease of alignment during erection, may decrease because the flanges may not come together easily. •Permanent connection efficiency may decrease due to the necessary grouting of the column splices.

Table 5-3: **Results of critical factor analyses for configuration option 2, System 1.**

Additional Potential Advantages (compared to configuration option 1)	Additional Potential Disadvantages (compared to configuration option 1)
•Fabrication efficiency may significantly increase, due to the elimination of the bottom plate. •Erection performance, or stability and ease of alignment during erection, may increase with the use of the I-beam girder configuration instead of the box-beam girder configuration.	•Fabrication efficiency may decrease, due to extra fabrication activities associated with the connection of the spanning elements (e.g., coping of the beams). •Preassembly efficiency may decrease, due to a slight increase the difficulty of attaching the spanning elements, due to increased congestion in the work area.

Table 5-4: Results of critical factor analyses for configuration option 3, System 1.

Additional Potential Advantages (compared to configuration option 1)	Additional Potential Disadvantages (compared to configuration option 1)
•Erection efficiency may increase, similar to option 1, due to the alignment mechanism provided by the angles (rather than the plates).	•Preassembly efficiency may decrease significantly, due to the need to attach the angles, which would require the panels to be raised. •Preassembly efficiency may decrease, due to the strict tolerances required, associated with the angles. •This configuration (discontinuous columns) may not be structurally sound.

Table 5-5: Results of critical factor analyses for configuration option 4, System 1.

Additional Potential Advantages (compared to configuration option 1)	Additional Potential Disadvantages (compared to configuration option 1)
•Erection efficiency may increase, similar to option 1, due to the alignment mechanism provided by the angles (rather than the plates). •Fabrication efficiency may increase significantly over configuration option 3, due to the elimination of the bottom plate. •Erection performance, or stability and ease of alignment during erection, may increase with the use of the I-beam girder configuration instead of the box-beam girder configuration.	•Preassembly efficiency may decrease significantly, due to the need to attach the angles, which would require the panels to be raised. •Preassembly efficiency may decrease, due to the strict tolerances required, associated with the angles. •Fabrication efficiency may decrease, due to extra fabrication activities associated with the connection of the spanning elements (e.g., coping of the beams). •Preassembly efficiency may decrease, due to a slight increase in the difficulty of attaching the joists, due to increased congestion in the work area. •This configuration (discontinuous columns) may not be structurally sound.

Table 5-6: Results of critical factor analyses for configuration option 5, System 1.

Additional Potential Advantages (compared to configuration option 1)	Additional Potential Disadvantages (compared to configuration option 1)
•Erection efficiency may increase, similar to option 1, due to the alignment mechanism provided by the ATLSS Connectors (rather than the plates). •Fabrication efficiency may increase, due to the elimination of the alignment plate. •Erection efficiency may increase, due to the use of multi-story columns.	•Transportation efficiency may slightly decrease, due the use of multi-story columns.

Table 5-7: Results of critical factor analyses for configuration option 6, System 1.

Additional Potential Advantages (compared to configuration option 1)	Additional Potential Disadvantages (compared to configuration option 1)
•Fabrication efficiency may increase, due to the elimination of the alignment plate.	•There is no alignment mechanism in this option, which may decrease erection efficiency.

Table 5-8: Results of critical factor analyses for configuration option 7, System 1.

Additional Potential Advantages (compared to configuration option 1)	Additional Potential Disadvantages (compared to configuration option 1)
•Fabrication efficiency may increase, due to the elimination of the alignment plate. •Fabrication efficiency may increase significantly, due to the elimination of the bottom plate. •Erection performance, or stability and ease of alignment during erection, may increase with the use of the I-beam girder configuration instead of the box-beam girder configuration.	•There is no alignment mechanism in this option, which may decrease erection efficiency. •Fabrication efficiency may decrease, due to extra fabrication activities associated with the connection of the spanning elements (e.g., coping of the beams). •Preassembly efficiency may decrease, due to a slight increase the difficulty of attaching the joists, due to increased congestion in the work area.

5.2.3 Structural Analysis of System 1

In this section, we describe the results of the structural analysis that we perform on a building using configuration options 1 and 2 of System 1. We perform this basic structural analysis to get an idea of whether these variations of the new system offer the potential to work and to provide information for the project analysis described in section 5.2.4. The analysis procedure involves calculating the required capacities of the different structural elements (e.g., girders, columns, connections) for a prototype building (see Figure D-1) designed with respect to typical gravity loads (see Table D-2), and determining whether the available structural members (e.g., channels, concrete-filled tubular columns) have sufficient strengths for use in the specified configurations. We describe this structural analysis procedure in detail, including calculations, in Appendix D.

First, we design the columns. Table 5-9 summarizes the required axial strength, resulting column, resulting capacity, and resulting weight of the columns, which are the same regardless of the framing option chosen.

Table 5-9: Structural analysis results for columns.

P_u	529 *k*
Resulting column	12x8 tube, ⅜ *in* thick
ϕP_n	615 *k*
Weight/ft	47.90 *lb*

To design the channel girder sections, we must first choose a section that has sufficient capacity before the concrete cures when only dead loads will be applied to the structure. Table 5-10 summarizes the required shear and moment capacities, the resulting section, and the resulting capacities for both configuration options and for the two frame options before the concrete cures.

Table 5-10: Structural analysis results for girders before concrete cures.

	Configuration Option 1		**Configuration Option 2**	
	Frame A	**Frame B**	**Frame A**	**Frame B**
M_u^+	213 *k-ft*	256 *k-ft*	213 *k-ft*	256 *k-ft*
V_u	34.1 *k*	34.1 *k*	34.1 *k*	34.1 *k*
Resulting section	MC13x31.8, 0.5 *in* plate	C15x40, 0.5 *in* plate	C15x33.9	MC13x50
ϕM_n	221 *k-ft*	280 *k-ft*	227 *k-ft*	261 *k-ft*
ϕV_n	190 *k*	304 *k*	234 *k*	398 *k*

Next, we find the required capacity of the channel girder section when the concrete is cured and all the loads will be applied to the structure (dead and live loads), and we check the capacities of the section chosen based on the noncomposite strength requirement. Table 5-11 summarizes the required shear and moment capacities, the section to be checked from the noncomposite analysis, and the resulting capacities for both configuration options and for the two frame options.

Table 5-11: Structural analysis results for girders after concrete cures.

	Configuration Option 1		Configuration Option 2	
	Frame A	Frame B	Frame A	Frame B
M_u^+	370 *k-ft*	444 *k-ft*	370 *k-ft*	444 *k-ft*
V_u	59.3 *k*	59.3 *k*	59.3 *k*	59.3 *k*
Checking section	MC13x31.8, 0.5 *in* plate	C15x40, 0.5 *in* plate	C15x33.9	MC13x50
ϕM_n	557 *k-ft*	706 *k-ft*	489 *k-ft*	597 *k-ft*
ϕV_n	190 *k*	304 *k*	234 *k*	398 *k*

Then we determine the required number and size of the spanning elements in the preassembled panels. We choose to use open web bar joists for the spanning elements, and we must redesign the channel girders to accommodate the depth of the joists. Table 5-12 summarizes the number and designations of joists we use, as well as the weights of the joists and redesigned channel sections for both configuration options and for the two frame options. Once we determine the number and sizes of the joists and redesigned channels, we calculate the weights of the panels, which are summarized in Table 5-13 for the various panel options.

Table 5-12: Joist results.

	Configuration Option 1		Configuration Option 2	
	Frame A	Frame B	Frame A	Frame B
# of joists	9	7	9	7
Joist designation	16K9	16K9	16K9	16K9
Weight of joists	10 *lb/ft*	10 *lb/ft*	10 *lb/ft*	10 *lb/ft*
Redesigned channels	MC18x42.7	MC18x42.7	MC18x42.7	MC18x42.7

Table 5-13: Panel weights.

	Configuration Option 1		Configuration Option 2	
	Frame A	Frame B	Frame A	Frame B
Panel weight	5.2 *tons*	4.7 *tons*	4.8 *tons*	4.3 *tons*

After the columns and girders are chosen, it is necessary to design the girder to column connection (panel to column connection). Table 5-14 summarizes the resulting connection for each of the configuration and frame options.

Table 5-14: Girder to column connection results.

	Configuration Option 1		Configuration Option 2	
	Frame A	Frame B	Frame A	Frame B
Required Seating Angle	L6x4x¾ 0'-9.5" long	L6x4x¾ 0'-8.5" long	L6x4x¾ 0'-8.3" long	L6x4x¾ 0'-10.3" long
Required Weld size, a	⅜ *in*	⅜ *in*	⅜ *in*	⅜ *in*

From this structural analysis, it is apparent that the available structural members do have sufficient capacities to carry the loads prescribed for this system. Table 5-15 summarizes the resulting columns, channel girders, bottom plates (if applicable), joists, and seating angles that can be used, as well as the panel weights for the two configuration options in this system, for the two framing options. It should be noted that there are no reasons to assume that the other panel options will not work (with the exception of panel option 4), since these are standard elements and configurations.

Table 5-15: Summary of members and connection elements used in System 1.

	Configuration Option 1		Configuration Option 2	
	Frame A	Frame B	Frame A	Frame B
Columns	12x8 tube ⅜ *in* thick	12x8 tube ⅜ *in* thick	12x8 tube ⅜ *in* thick	12x8 tube ⅜ *in* thick
Channels	MC18x42.7	MC18x42.7	MC18x42.7	MC18x42.7
Bottom plates	0.5 *in* thick	0.5 *in* thick	N/A	N/A
Joists	9--16K9	7--16K9	9--16K9	7--16K9
Panel weight	5.2 *tons*	4.7 *tons*	4.8 *tons*	4.3 *tons*
Seating angles	L6x4x¾	L6x4x¾	L6x4x¾	L6x4x¾

5.2.4 Project Analysis of System 1

In this section, we describe the results of the project analysis that we perform on a building using configuration option 2 in System 1, which seems to be the most practical option from the structural analysis (no bottom plate is necessary). We also use bar joist spanning elements (panel option 1). We assume that the channel girders are running in the short direction of the prototype building (Frame A--see Figure D-2), and the crew consists of nine workers (including a supervisor and a helper) that are distributed according to the activities being accomplished. From this analysis, we determine the approximate duration and worker air time required to construct the building. We also conduct a project analysis for a traditionally constructed system, to provide a basis for comparison. We developed the methodology for estimating project durations of partially preassembled buildings from the site observations and data collected from the DuPont construction site, which we discussed in section 4.1. This methodology involves using a very large spreadsheet to calculate the required time for each individual activity and the total elapsed time for construction. The information required for this process includes sizes and numbers of members, panels, and connections, which we calculated from the structural analysis, as well as an idea of the project's flow. In Appendix E, we describe this detailed analysis process. The results of this analysis are shown in Tables 5-16 and 5-17.

Table 5-16: Comparison of System 1 to traditionally constructed building--duration.

Building	Duration	% Increase
Traditional	24.3 days	-----
System 1	30.7 days	26%

Table 5-17: Comparison of System 1 to traditionally constructed building--worker air time.

Building	Worker Air Time	% Decrease
Traditional	88.1 days	-----
System 1	28.8 days	67%

As shown in Tables 5-16 and 5-17, from this project analysis, the duration of System 1 actually increases when compared to that of a traditionally constructed building, although the worker air times decrease significantly. We discuss possible reasons for the increase in duration, as well as implications of these results, in section 5.5.

5.3 System 2: "Stub Column System"

5.3.1 Objectives and General Description of System 2

The "Stub Column System" constitutes a complete structural framing system which consists of vertical load-bearing elements, horizontal load-bearing elements, and floor surfaces. The cast-in-place concrete floor slab produces composite action with the steel elements.

In developing this system, our objectives were to move the field connections away from the points of highest complexity (i.e., the beam to column connection), to move the field connections away from the highest moment regions, to incorporate the decking as part of the preassembled panel, to use repetitive bay-size panels, while minimizing member redundancy, and to use standard structural elements.

The vertical elements in System 2 are single or multi-story W-shape or tubular steel columns with welded or rigidly bolted beam studs in the transverse direction (Figure 5-22). The columns can use any of the available acceptable techniques for column to column connections. The beam stubs, with lengths designed to put the panel to stub splice connection at the moment inflection point, have a preattached bottom plate that provides a seat for the panels to rest upon and forms part of the splice connection. Seating angles or clip angles (depending on the spanning elements used) are preattached to each column stub to provide a means for the connection of the stick-built spanning elements that run parallel to the spanning elements of the panels.

The horizontal elements are preassembled panels for spanning among four columns (one bay). The perimeter elements of the panels in the transverse directions are channel members that span between the beam stubs (Figure 5-23). The channels are placed with the web of the channel on the exterior edge of the panel so the two channels from adjacent panels form an I-beam configuration with a double web (Figure 5-24). The channels of adjacent panels are bolted together at intervals along their length between the columns to increase torsional resistance. The channels may also be connected with a bottom plate, if this is required to provide sufficient strength and stability of the girders. (For the loads in our example application, discussed in section 5.3.3, we find that this bottom plate is not necessary.) A plate is attached to the top of each channel at each corner of the panel which forms part of the panel to column connection. This is a one-way system; there are no girders spanning from column to column in the direction parallel to the spanning elements.

After the columns and stick-built beams surrounding a particular panel are set, the panel can be erected. The panel rests on and is connected to the bottom plates which are preattached to the beam stubs, and the plates that are attached to the top of the channels are connected to the top of the beam stubs. The corrugated metal decking, which is preattached to the panel during erection, rests on and is connected to the stick-built spanning elements.

Once two adjacent panels are set, the splice connection can be completed with a web splice plate that connects the webs of both channels to the beam stub, and the adjacent channels are joined at intervals through the webs.

There are four options for the type of spanning elements in the preassembled panels, described in section 5.3.1.1 (see Table 5-1). The preassembled panel also includes corrugated metal decking and shear studs (with the exception of innovative panel option 4). A cast-in-place concrete floor slab produces composite action with the steel elements.

5.3.1.1 Panel Options for System 2

Panel Option 1: (Open-Bar Joist Spanning Elements)

In panel option 1, open-bar joists span from one channel to the opposite parallel channel. Corrugated metal decking covers the complete surface of the panel, and shear studs are placed along the tops of the channels and along the joist line. The bar joists are connected to the channel webs with angles that are preattached to the channels, and they are slightly lower than the top of the channel (Figure 5-14). The open-bar joists have to be designed and fabricated to specific span lengths.

Panel Option 2: (Standard W-Shape Beam Spanning Elements)

In panel option 2, standard W-shape beams span from one channel to the opposite parallel channel. Corrugated metal decking covers the complete surface of the panel, and shear studs are placed along the tops of the channels and along the beam line. The beams, which must be coped, are connected to the channel web with a shear angle (Figure 5-16). The W-shape beams do not require special design or fabrication requirements for specific lengths but can be cut to length from common stock.

Panel Option 3: (Castellated Beam Spanning Elements)

In panel option 3, castellated beams span from one channel to the opposite parallel channel. Corrugated metal decking covers the complete surface of the panel, and shear studs are placed along the tops of the channels and along the joist line. The castellated beams, which must be coped, are connected to the channel web with a shear angle (Figure 5-18). The castellated beams do not require special design or fabrication requirements for specific lengths but can be cut to length from common stock.

Panel Option 4: ("Modified" Castellated Beam Spanning Elements)

In panel option 4, castellated beams with an additional half of the castellation welded along the top flange span from one channel to the opposite parallel channel. Corrugated metal decking is placed between the top flanges of the castellated beams so that the additional half-castellation protrudes above the decking to act as a shear element for a composite deck, replacing the need for shear studs along the spanning elements (Figure 5-19). This panel option is an innovative concept which has never been attempted and must be tested thoroughly before it can actually be put into practice (it may not be feasible). In theory, the "modified" castellated beams, which must be coped, are connected to the channel web with a shear angle (Figure 5-21). The castellated beams do not require special design or fabrication requirements for specific lengths but can be cut to length from common stock.

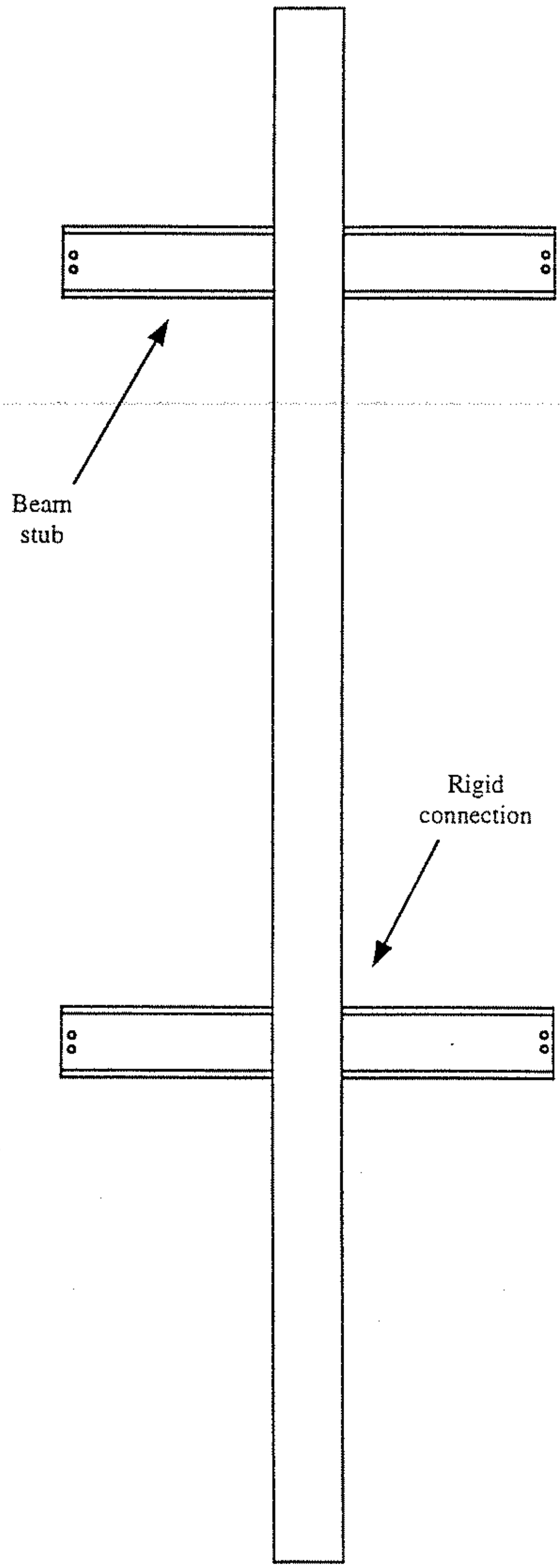


Figure 5-22: Typical Column

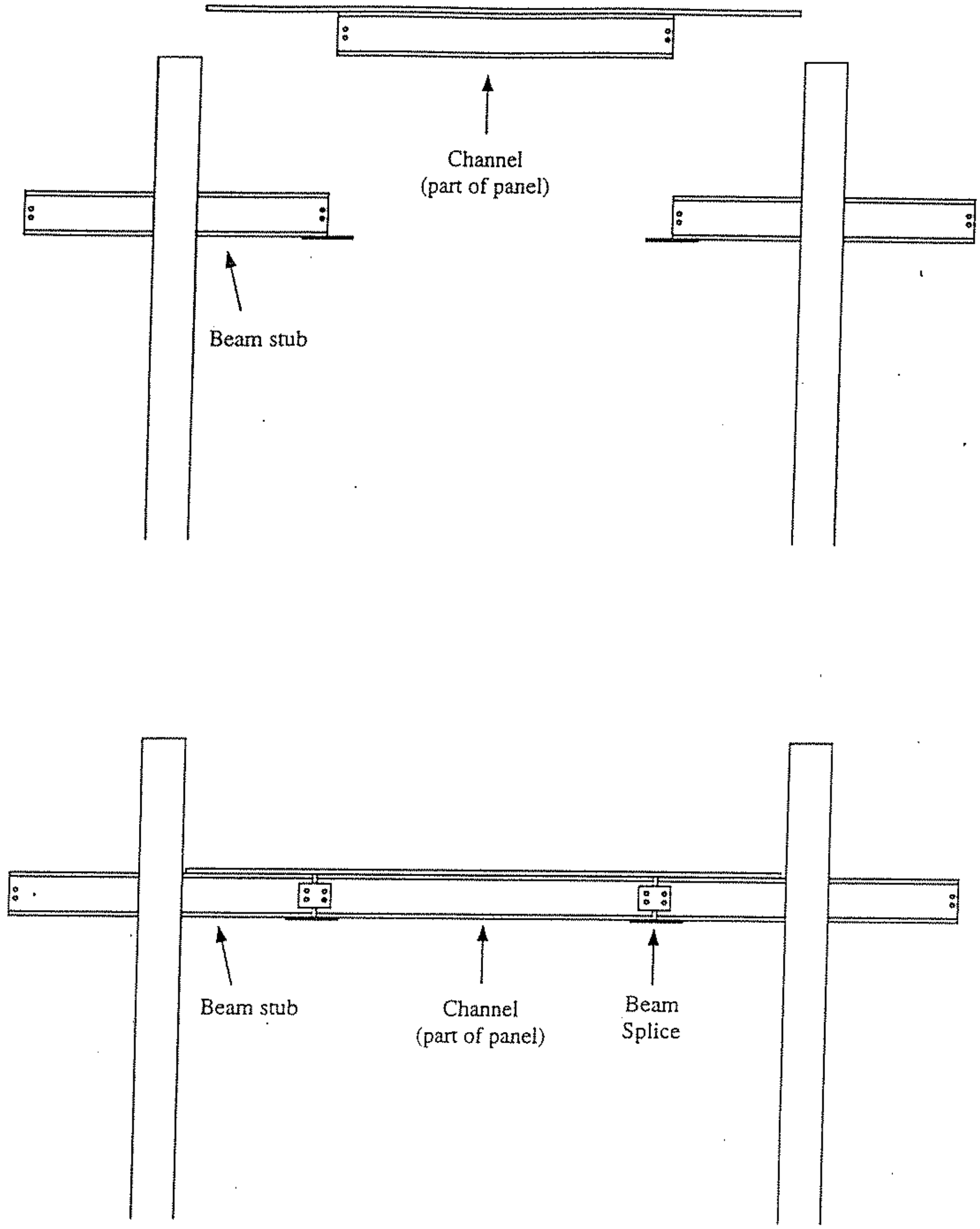
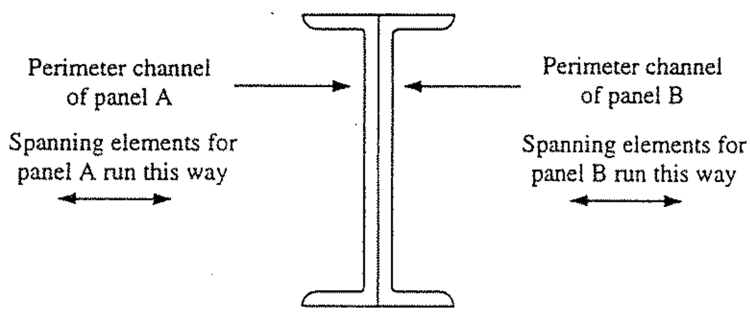


Figure 5-23: Illustration of how perimeter elements of panels span between beam stubs.



(Spanning elements, metal decking, and concrete slab not shown.)

Figure 5-24: Channel girder configuration.

5.3.2 Analysis of System 2 with Respect to Critical Factors

Before we perform a structural performance analysis or project impact analysis on System 2, we perform an analysis with respect to the critical factors to determine whether the system is effectively designed for preassembly. In this section, we describe the results of this analysis, noting the advantages and disadvantages of the new system when compared to traditionally constructed systems. In performing this analysis, we are comparing the activities that we assume would be involved in constructing the new system to the activities associated with traditional construction. When making this comparison, we notice that there are some activities involved in the construction of the new system that are not included in traditionally constructed systems (e.g., preassembly); however, there are also activities that as a result of using the new system are more efficient or safer than if the system were traditionally constructed (e.g., erection). These are the types of tradeoffs that this analysis brings out.

Table 5-18 shows the results of this critical factor analysis, the potential advantages and disadvantages of this system. It should be noted that for System 2, redundant members are used (i.e., two channels rather than one W-shape beam); however, they are combined in such a way as to take advantage of their redundancy (two smaller members combine to provide the strength of one larger member). This may or may not be a disadvantage, but it not listed in Table 5-18. Other disadvantages not incorporated in Table 5-18 include the possibility of needing a larger crane to lift the heavy panels than would be required for traditional construction and the absence of an alignment mechanism in this system.

Table 5-18: Results of critical factor analysis for System 2.

Potential Advantages	Potential Disadvantages
•Erection efficiency may increase, due to the need to erect significantly fewer members, when compared to traditional construction. •Worker safety during erection may significantly increase, due to fewer labor hours, especially in the air, and lower danger exposure. •Permanent connection efficiency may increase, due to the transfer of much of the permanent connection stage to the ground during preassembly, instead of in the air after erection. •Worker safety during permanent connection may significantly increase, due to the transfer of much of the permanent connection stage to the ground during preassembly, instead of in the air after erection. •Decking efficiency may increase, due to the transfer of many of the decking activities (decking installation, shear studs) to the ground during preassembly, instead of in the air after erection. •Worker safety during decking may significantly increase, due to the transfer of many of the decking activities (decking installation, shear studs) to the ground during preassembly, instead of in the air after erection.	•Fabrication efficiency may decrease, due to (1) the increase in the number of members (due to the redundant channels) when compared to traditional construction and (2) the need for strict tolerances. •Transportation efficiency may decrease, due to the need to transport the stub columns. •Preassembly efficiency may decrease, due to the need for strict tolerances, associated with the alignment of the panel during erection. •Erection efficiency may decrease, due to the larger and heavier units to be erected, when compared to traditional construction, as well as the fact that there are more connections per unit than stick-built members, and it is more difficult to balance the larger units.

5.3.3 Structural Analysis of System 2

In this section, we describe the results of the structural analysis that we perform on a building using System 2. We perform this basic structural analysis to get an idea of whether this new system offers the potential to work and to provide information about the member sizes and connection types for the project analysis described in section 5.3.4. The same analysis procedure is used as was used for System 1. We describe this structural analysis procedure in detail, including calculations, in Appendix D.

First, we design the columns. Table 5-19 summarizes the required axial strength, resulting column, and resulting capacity of the columns, which are the same regardless of the framing option chosen.

Table 5-19: Structural analysis results for columns.

P_u	529 k
Resulting column	W12x79
ϕP_n	599 k

To design the beam stub and channel girder sections, we must first choose sections that have sufficient capacity before the concrete cures when only dead loads will be applied to the structure. Tables 5-20 and 5-21 summarize the required shear and moment capacities, the resulting section, and the resulting capacities of the beam stub and noncomposite channel section, respectively, for the two frame options before the concrete cures.

Table 5-20: Structural analysis results for beam stubs before concrete cures.

	Frame A	Frame B
M_u^-	142 k-ft	170 k-ft
V_u	34.1 k	34.1 k
Resulting section	W14x38	W14x48
ϕM_n	148 k-ft	190 k-ft
ϕV_n	85.0 k	91.1 k

Table 5-21: Structural analysis results for channel girders before concrete cures.

	Frame A	Frame B
M_u^+	71.1 k-ft	85.3 k-ft
V_u	34.1 k	34.1 k
Resulting section	C10x15.3	MC8x22.8
ϕM_n	73 k-ft	86 k-ft
ϕV_n	93 k	133 k

Next, we find the required capacity of the beam stub and composite channel girder sections when the concrete is cured and all the loads will be applied to the structure (dead and live loads), and we check the capacities of the members chosen based on the noncomposite strength requirement. Table 5-22 summarizes the required shear and moment capacities for the beam stubs, as well as the resulting section and capacities, for the two frame options. Table 5-23 summarizes the required shear and moment capacities for the channel sections, the sections to be checked from the noncomposite analysis, and the resulting capacities, for the two frame options. From this analysis, it was necessary to redesign the beam stubs, although the channel sections chosen based on the noncomposite strength requirement are satisfactory. The channel sections may have to be sized up to the depth of the beam stub, however, if a bottom plate is desired.

Table 5-22: Structural analysis results for beam stubs after concrete cures.

	Frame A	Frame B
M_u^-	247 *k-ft*	296 *k-ft*
V_u	59.3 *k*	59.3 *k*
Resulting section	W14x68	W14x74
ϕM_n	278 *k-ft*	302 *k-ft*
ϕV_n	113 *k*	124 *k*

Table 5-23: Structural analysis results for channel girders after concrete cures.

	Frame A	Frame B
M_u^+	123 *k-ft*	148 *k-ft*
V_u	59.3 *k*	59.3 *k*
Checking section	C10x15.3	MC8x22.8
ϕM_n	179 *k-ft*	219 *k-ft*
ϕV_n	93 *k*	133 *k*

Then we determine the required number and size of the spanning elements in the preassembled panels. We choose to use open web bar joists for the spanning elements, and we must redesign the channel girders to accommodate the depth of the joists. Table 5-24 summarizes the number and designations of joists we use, as well as the weights of the joists and redesigned channel sections for the two frame options. Once we determine the number and sizes of the joists and redesigned channels, we calculate the weights of the panels, which are summarized in Table 5-25 for the two frame options.

Table 5-24: Joist results.

	Frame A	Frame B
Total # of joists	9	7
# of joists per panel	5	3
# of stick-built joists per bay	4	4
Joist designation	16K9	16K9
Weight of joists	10 *lb/ft*	10 *lb/ft*
Redesigned channels	MC18x42.7	MC18x42.7

Table 5-25: Panel weights.

	Frame A	Frame B
Panel weight	2.7 *tons*	2.2 *tons*

After the columns and girders are chosen, it is necessary to design the splice between the beam stub and the channel section. For both of the frame options, two bolts on each side of the beam splice are required to connect the beam stub and channel sections. The beam stubs must be sized up to the depth of the redesigned channels to use a bottom plate. Table 5-26 summarizes the resulting connection and redesigned beam stub sections.

Table 5-26: Girder splice connection results.

	Frame A	Frame B
# of bolts	2	2
Redesigned beam stubs	W18x55	W18x60

From this structural analysis, it is apparent that the available structural members do have sufficient capacities to carry the loads prescribed for this system. Table 5-27 summarizes the resulting columns, beam stubs, channel girders, joists (part of panel and stick-built), and the number of bolts in the beam splice that can be used for the two configuration options in this system, for the two framing options. As with System 1, it should be noted that there are no reasons to assume that the other panel options will not work (with the exception of panel option 4), since these are standard elements and configurations.

Table 5-27: Summary of members and connection elements used in System 2.

	Frame A	Frame B
Columns	W12x79	W12x79
Beam stubs	W18x55	W18x60
Channels	MC18x42.7	MC18x42.7
Total # joists	9	7
# joists per panel	5	3
# joists per stick-built bay	4	4
Panel weight	2.7 *tons*	2.2 *tons*
Bolts in girder splice	2 bolts	2 bolts

5.3.4 Project Analysis of System 2

In this section, we describe the results of the project analysis that we perform on a building using System 2, with bar joist spanning elements (panel option 1). We use the same procedure and assumptions as we used for the project analysis of System 2, and we determine the approximate duration and worker air time required to construct the building, in addition to a traditionally constructed building. We describe this detailed analysis process in Appendix E. The results of this analysis are shown in Tables 5-28 and 5-29.

Table 5-28: Comparison of System 2 to traditionally constructed building--duration.

Building	Duration	% Increase
Traditional	24.3 days	-----
System 2	32.7 days	35%

Table 5-29: Comparison of System 2 to traditionally constructed building--worker air time.

Building	Worker Air Time	% Decrease
Traditional	88.1 days	-----
System 2	61.1 days	31%

As shown in Tables 5-28 and 5-29, from this project analysis, the duration of System 2 actually increases when compared to that of a traditionally constructed building, although the worker air times decrease significantly. We discuss possible reasons for the increase in duration, as well as implications of these results, in section 5.5.

5.4 System 3: "Flared Column System"

5.4.1 Objectives and General Description of System 3

The "Flared Column System" is a complete structural framing system which consists of vertical load-bearing elements, horizontal load-bearing elements, and floor surfaces. It incorporates special connections to enable erection with little manual alignment or attachment. The cast-in-place concrete for the panel connections creates a monolithic connection.

In developing this system, our objectives were to provide an enlarged bearing surface as part of the column for the panels to rest upon, to simplify connections and to make the system stable immediately after erection; to provide an alignment mechanism for the panels; to incorporate the decking as part of the preassembled panel; and to use repetitive bay-size panels, while minimizing member redundancy. This system differs most in terms of the standardization of members and configurations than the other two systems we discussed.

The vertical elements in System 3 have components which provide a stiffened surface on which the panels rest. The columns may be made of precast concrete or steel, and may be one-story or multi-story, depending on the option chosen. The two possible column options are described in section 5.4.1.1 (see Figures 5-25 and 5-26).

The horizontal elements are preassembled panels for spanning among four columns (one bay). Each panel consists of precast concrete on corrugated steel with steel reinforcement (Figure 5-27). A topping slab and cast in place connection are needed after erection of the panels. Voids are included in the columns at the level of the top of the panel, in configuration 1, or the bottom of the upper columns are spaced up off of the panel, in configuration 2, so reinforcing steel can be placed within the depth of the column in two directions (corresponding to the panel to panel connections). The cast-in-place concrete forms a monolithic connection through the column depth. Along the perimeter of each panel is an area of reduced depth without the full depth of the concrete. This area is used to place additional steel reinforcement and cast-in-place concrete to create a monolithic connection. Across the top of the panel to panel connection, within the topping slab depth, reinforcing mesh creates a transverse connection between panels.

5.4.1.1 Configuration Options for System 3

Configuration Option 1: (Multi-story steel or precast columns with steel collars.)

In configuration option 1, the columns can be single or multi-story, and can be made from steel or precast concrete. Cast steel collars (or alternatively fabricated collars) are fit over the columns at the location of the panel to column connections to provide a bearing area for the panels to rest on. The collars include protruding bolts that act as part of an alignment mechanism with the panels (Figure 5-25). In this option, all of the available acceptable techniques for column to column connections can be used. The columns must include a void at the level of the top of the panel for the placement of reinforcing steel within the depth of the column.

The protruding bolts in the column collars and corresponding holes in the panels provide an alignment mechanism, as well as part of the panel to column connection (Figure 5-28). During erection, the panels fit over these bolts and rest upon the column collar. Before the topping slab and connections are poured, these bolts permanently attach the panel to the column.

Configuration Option 2: (One story precast concrete "lily" columns.)

In configuration option 2, each one-story precast concrete column is flared at the top to provide a bearing surface to receive the preassembled panels (Figure 5-26). The flared elements also have embedded bolts that act as part of an alignment mechanism with the panels. The flared columns use a *Splice Sleeve*® column to column connection; the top of the lower column has protruding metal bolts, and the bottom of the upper column has receiving sleeves of metal tubes which fit over the bolts during erection. The columns are spaced far enough apart to allow the placement of reinforcing steel through the depth of the column in two directions. The connection is then made with cast-in-place concrete.

The embedded bolts in the columns and corresponding holes in the panels provide an alignment mechanism, as well as part of the panel to column connection (Figure 5-29). During erection, the panels fit over these bolts and rest upon the column flare. Before the topping slab and connections are poured, these bolts permanently attach the panel to the column.

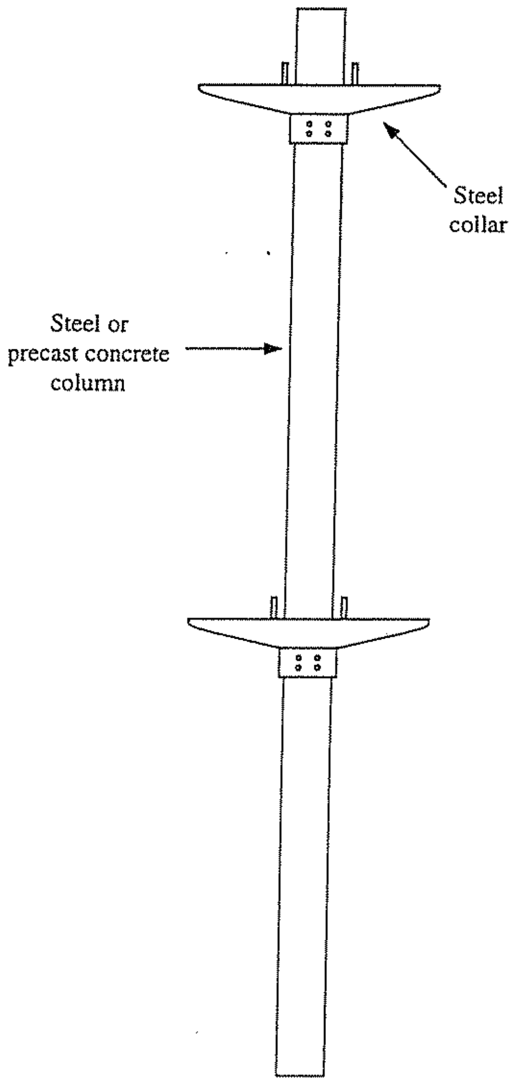


Figure 5-25: Typical column, Option 1.

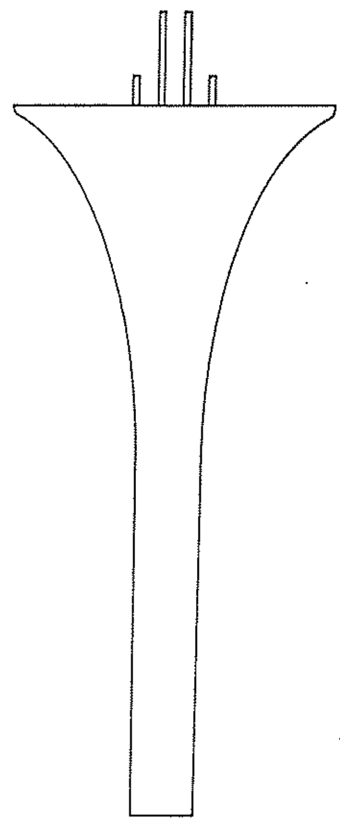


Figure 5-26: Typical column, Option 2.

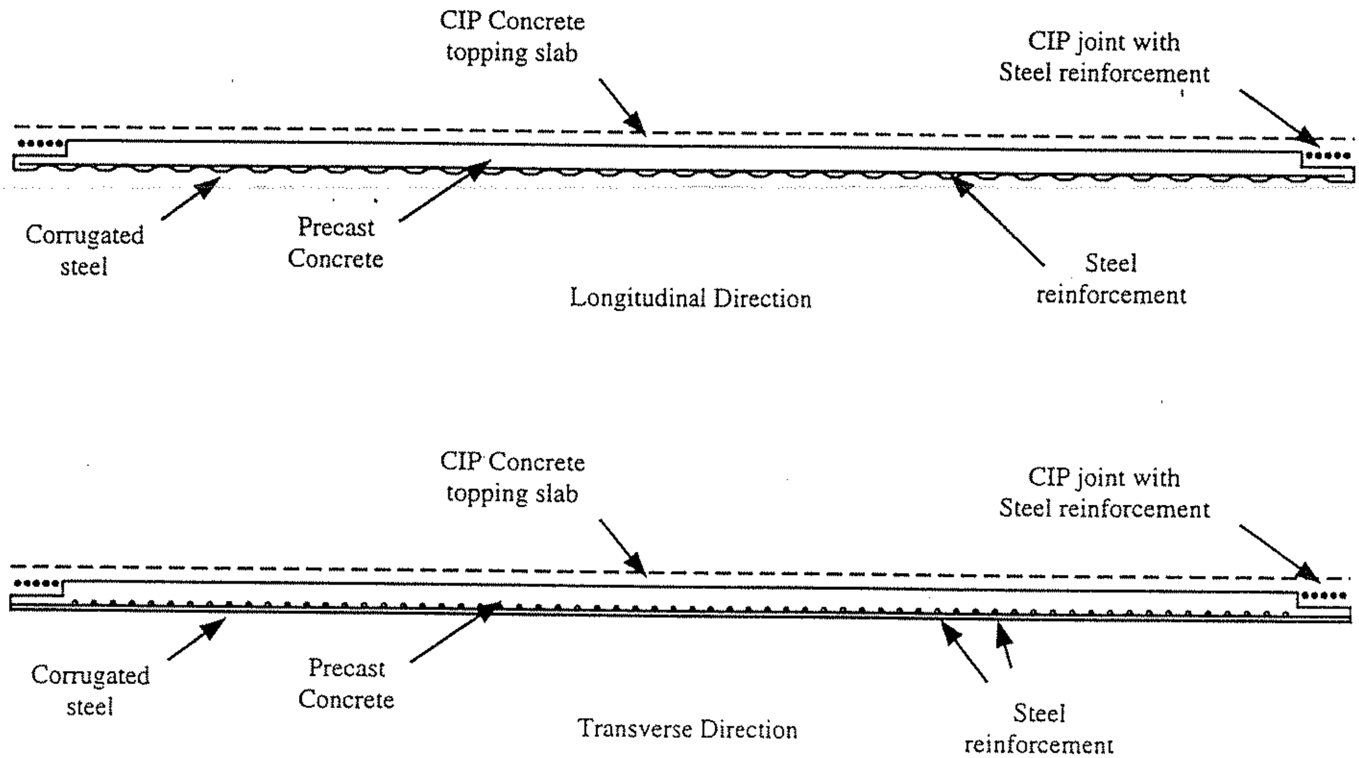


Figure 5-27: Cross-section of typical panel.

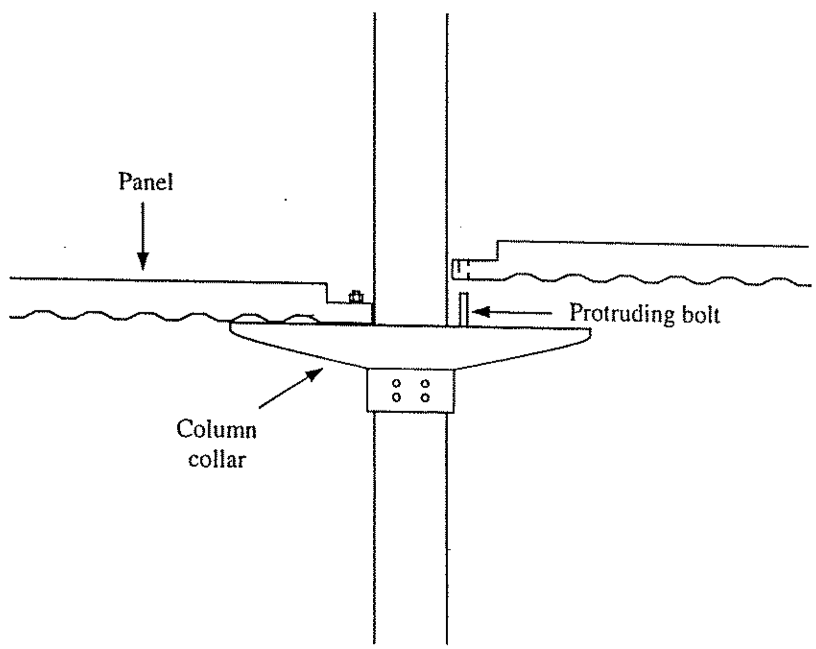


Figure 5-28: Overall system view, Option 1.

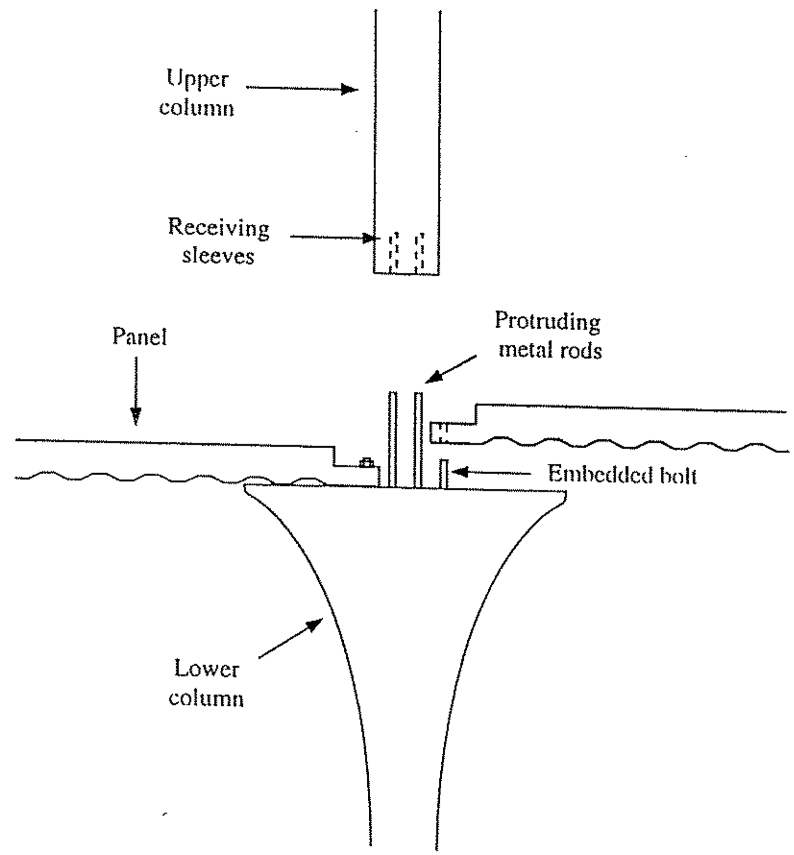


Figure 5-29: Overall system view, Option 2.

5.4.2 Analysis of System 3 with Respect to Critical Factors

As with Systems 1 and 2, we perform an analysis with respect to the critical factors for System 3 to determine whether the system is effectively designed for preassembly. In this section, we describe the results of the analyses that we perform on buildings using the variations of System 3. This analysis helps to determine whether the systems are effectively designed for preassembly and brings out the advantages and disadvantages of the new systems, when compared to traditionally constructed systems. We use this analysis to determine whether the variations of this system have inherent flaws that might cause major problems during construction and to identify tradeoffs between the different types of construction. In performing this analysis, we are comparing the activities that we assume would be involved in constructing the new systems to the activities associated with traditional construction. When making this comparison, we notice that there are some activities involved in the construction of the new systems that are not included in traditionally constructed systems (e.g., preassembly); however, there are also activities that as a result of using the new system are more efficient or safer than if the system were traditionally constructed (e.g., erection). These are the types of tradeoffs that this analysis brings out.

Table 5-30 shows the results of the critical factor analysis for configuration option 1, the potential advantages and disadvantages of this system. The results are similar for configuration option 2, so in Table 5-31 we list how this configuration option differs from configuration option 1, and whether the differences are advantages or disadvantages, rather than repeating similar results. One disadvantage not incorporated in Table 5-30 is the possibility of needing a larger crane to lift the heavy panels than would be required for traditional construction. Another issue is that the nature of the permanent connection changes with this system, when compared to traditional construction; although there are no bolted connections, a lot of work is required to set rebar to make the panel to panel and panel to column connections. Therefore, it is difficult to determine how the permanent connection efficiency and safety are affected.

Table 5-30: Results of critical factor analysis for configuration option 1, System 3.

Potential Advantages	Potential Disadvantages
•Fabrication efficiency may significantly increase, due to the decrease in the number of members that need to be fabricated (the panels are assembled in the field).. •Erection efficiency may increase, due to (1) the alignment mechanism and (2) the need to erect significantly fewer members, when compared to traditional construction. •Worker safety during erection may significantly increase, due to fewer labor hours, especially in the air, and lower danger exposure. •Decking efficiency may increase, due to the transfer of many of the decking activities to the ground during preassembly, instead of in the air after erection. •Worker safety during decking may significantly increase, due to the transfer of many of the decking activities to the ground during preassembly, instead of in the air after erection.	•Design efficiency may decrease, due to the nonstandard structural configuration, which may require special design consideration. •Fabrication efficiency may significantly decrease, due to the need to fabricate nonstandard column collars. •Transportation and handling efficiencies may decrease, due to the difficulties stacking the columns if the collars are preattached. •Preassembly efficiency may decrease, due to the need for strict tolerances, associated with the alignment of the panel during erection. •Erection efficiency may decrease, due to the larger and much heavier units to be erected, when compared to traditional construction.

Table 5-31: Results of critical factor analyses for configuration option 2, System 3.

Additional Potential Advantages (compared to configuration option 1)	Additional Potential Disadvantages (compared to configuration option 1)
None	•Performance of the members during transportation and handling may significantly decrease, due to the fragile "lily" columns. •Transportation and handling efficiencies may decrease, due to the weight and difficulty stacking the columns.

5.4.3 Structural Analysis of System 3

Due to the complexity and nonstandard nature of System 3, we did not perform a structural analysis for this system. Therefore, this system is not proven to be technically feasible.

5.4.4 Project Analysis of System 3

Because we did not perform a structural analysis on System 3, which is needed to provide information for the project analysis, and since we do not have any production rates for this type of system, a project analysis of System 3 is beyond the scope of this report.

5.5 Conclusions

Through the application of the critical factor measures (discussed in section 4.2) to the new systems, we determine which variations of the new systems would be most practical and identify several advantages and disadvantages that are common among the three preassembled systems. Table 5-32 shows the variations of the new systems that we find to be most practical and that we perform structural analyses on. Common advantages to the three systems include a possible increase in erection efficiency due to the need to erect significantly fewer members, possible increases in permanent connection and decking efficiencies due to the transfer of many of the permanent connection and decking activities to the ground during preassembly instead of in the air after erection, and possible increases in worker safety during erection, permanent connection, and decking due to the transfer of many of the activities to the ground during preassembly instead of in the air. Common disadvantages include a possible decrease in design efficiency due to the nonstandard structural configurations and a possible decrease in erection efficiency due to the larger and heavier units to be erected, when compared to traditional construction, as well as the fact that it is more difficult to balance and align the large units. Additional factors that must be considered with all of the new systems include the possible need for a larger crane than would be necessary for traditional construction, and the fact that the extra activity of preassembly is added, a significant activity that involves many processes (e.g., erection, permanent connection, and decking).

Table 5-32: Most practical variations of the new systems.

System 1	1) Box-beam girder configuration, plate alignment mechanism with column rods, one-story columns connected with splice sleeves 2) I-beam girder configuration, plate alignment mechanism with column rods, one-story columns connected with splice sleeves
System 2	I-beam girder configuration, multi-story steel columns with rigid beam stubs in transverse direction, standard splice connection.

From the structural analyses that we perform on the systems, we find that Systems 1 and 2 are technically feasible for the loads we impose on the systems, although we only considered basic gravity loads, assuming that appropriate lateral support such as bracing or additional reinforcement of the connections will be provided. Due to the complexity of System 3, we do not perform a structural analysis, and therefore the technical feasibility of this system is not proven. It should be noted that for Systems 1 and 2, the channel girders must be sized up significantly to accommodate the depth of bar joists, which may not be cost effective.

From the project analyses that we perform on the systems, we find that both Systems 1 and 2 provide the opportunity to increase safety of the workers, as shown in Table 5-33, although they would both likely increase the total duration of the project, as shown in Table 5-34. Possible reasons for this increase in duration include alignment difficulties, problems with member redundancies, and the incompatibility of standard connections to this type of construction. Future research dealing with these problems might substantially improve the benefits preassembly can offer.

Table 5-33: Comparison of new systems to traditionally constructed building--duration.

Building	Duration	% Increase
Traditional	24.3 days	-----
System 1	30.7 days	26%
System 2	32.7 days	35%

Table 5-34: Comparison of new systems to traditionally constructed building--worker air time.

Building	Worker Air Time	% Decrease
Traditional	88.1 days	-----
System 1	28.8 days	67%
System 2	61.1 days	31%

Chapter 6: Discussion

Based on the insights gained from the site observations, discussed in section 4.1, we develop conceptual designs of innovative structural systems that might improve project results, as well as two methods to determine the effectiveness of these types of systems, the application of the critical factors of structural systems that we identify and the methodology for estimating construction durations. We use the new systems to test the usefulness of these methods, as well as performing structural analyses on the systems to check their technical feasibility. It is also hoped that these concepts may provide ideas which lead to improvements in the economic efficiency and construction effectiveness of preassembly methods, although from our analyses, it is questionable whether these systems will ever actually be put into practice.

Since we use the results from the observation of one preassembled site to base many of our results, the reliability of these results may be questionable. Production rates and the flow of activities do vary from site to site, and site conditions play a very important role in these issues. However, the flow of activities that we assume in our analyses does take into account the distribution of limited resources (e.g., workers and crane) and considers possible site constraints (e.g., limited assembly area), so the general trend given by our results are likely accurate for typical projects, without extreme conditions. In addition, we varied the production rates that we used in the duration analysis, and the project results did not change significantly.

The critical factor measures that we identify provide an objective way to determine how the characteristics of a particular preassembled system affect the different phases of the project, by bringing out the potential advantages and disadvantages of the system. Through the application of these critical factors to the new systems, we determine which variations of the new systems would be most practical as well as advantages and disadvantages that are common to the three systems. While these critical factor measures provide useful information about specific projects, in terms of their advantages and disadvantages, by no means do they provide a quantitative measure of whether one particular project is more beneficial than another. While this type of measure would be extremely useful, the difficulty in objectively scaling the importance of the different activities, the dependency of many of the activities on site conditions, and the project-specific nature of the measures would make accuracy very difficult to achieve for this type of procedure.

The structural analyses that we perform on each system allow us to determine whether the systems are technically feasible and provide information for the project analyses. From these structural analyses, we find that Systems 1 and 2 are technically feasible for the loads we impose on the systems, although we only consider basic gravity loads, assuming that appropriate lateral support such as bracing or additional reinforcement of the connections will be provided. Actual testing and more detailed analyses must be done before any of these systems can be put into practice. Due to the complexity of System 3, we do not perform a structural analysis, and therefore the technical feasibility is not proven. Although we prove the technical feasibility of Systems 1 and 2, with respect to basic gravity loads, the economic feasibility is not at all proven; issues such as the need to size up the channels to accommodate the spanning elements are not taken into account in a cost assessment.

Through telephone interviews with structural designers, manufacturers, fabricators, and erectors, discussed in section 3.3, we get an idea of the types of issues that concern the industry members, with respect to the new systems. One general concern is that even if new systems such as these offer the potential to provide advantages such as faster or safer erection, they may not be able to compete with traditional systems that incorporate standard shapes that are much easier to obtain and configurations that are easier to design; therefore, new systems must be integrated into standard methodology in order to be competitive. Another major concern deals with the possibility of increased difficulty of erection due to possible tolerance problems. Although we attempt to provide oversized holes and alignment mechanisms with the new systems, better guiding mechanisms (e.g., funnel concept, ATLSS Connector) would likely be more effective. There was a general consensus among the industry members, however, in favor of the concept of including the decking as part of the preassembled panel, which is incorporated in our three systems.

There were various concerns expressed by the industry members dealing with System 1. One additional advantage that this system offers is fire endurance associated with the concrete-filled tubes, possibly eliminating the need to fire-proof the outside of the columns. The economy of using these one-story concrete-filled tubes, however, was questioned, as well as the need for the concrete inside the tubes, since this system is intended for low to mid rise buildings. An additional concern with this system is the need for reinforcement inside the tubes and a means for composite action between the tubes and the concrete (e.g., shear lugs), which would substantially increase cost. The weight of the tubes would increase transportation and handling difficulties, and if the concrete was poured on-site, work progress may be held up. The strength and durability of the column rods and the receiving sleeves were also questioned; one suggestion was to use a single tapered splice pin to join the columns, although this would eliminate the alignment mechanism we suggest with the column rods and the panels. An alternative suggestion for panel to column alignment was the use of ATLSS Connectors at the corners of the panels, which we incorporate in configuration option 5 of this system (discussed in Chapter 5). The channels in this system would probably have sufficient capacity, although there were concerns for the economy of this aspect, as well as an overwhelming preference for back to back channels (I-beam configuration) as opposed to flange tip to flange tip channels (box-beam configuration), due to stability, the cost associated with a bottom plate, and the logistics of pouring the concrete slab on a possibly discontinuous surface (where the top flanges of the channels meet in the box-beam configuration).

For System 2, there was a question about the economy of the moment connections for the stub columns, due to fabrication costs associated with this connection; an alternative is to make this connection pinned, since lateral stability may be provided with lateral bracing. Safety and alignment problems were also concerns with this system, with respect to the "wobbly" columns that the panels must rest on. Another concern is the transportation difficulty associated with these stub columns.

For System 3, there were concerns about the high cost of the column collars, as well as the fragileness, cost, and possible transportation and handling difficulties of the one-story precast "lily" columns. Although casting the column collars can be done without difficulty, it would probably be much more economical to fabricate a similar collar. There were many questions regarding the feasibility of the precast panel, concerning the difficulties in lifting and aligning such a heavy panel, as well as the practicality and labor intensive nature of the reinforcement.

Since the first three panel options for Systems 1 and 2 are standard, there were no concerns about these, although there was widespread concern with the feasibility of panel option 4, with the "modified" castellated beam spanning elements. Many of the industry members noted the need for a long-term study to determine the performance of the upper level of castellation for composite action, when compared to shear studs. We recognize this issue and introduce this option primarily as a concept that may eventually lead to a way to eliminate the need for shear studs, not by any means for immediate use. A representative from a manufacturer of castellated beams thinks that this is a promising idea that deserves further consideration.

The methodology that we develop for estimating construction duration and worker air time offers a quantitative way to identify the possibility of success or potential savings of a specific preassembled project, assuming the previously mentioned problems are overcome. From the project analyses that we perform on the new systems, discussed in Chapter 5, we find that both Systems 1 and 2 provide the opportunity to increase the safety of the workers, although they would both likely increase the total duration of the project. The increase in worker safety associated with the use of the systems, while very desirable, does not alone justify their use when the duration and cost of the project will likely increase. There are several possible causes for the increase in duration of the new systems. The need to align the members both in the preassembled unit and when connecting the unit to the rest of the building may increase the times required for preassembly. Tolerance problems often lead to difficulties with the latter and thus longer times are required for this process. The member redundancies in Systems 1 and 2, although structurally useful, may also increase the time due to an increased number of members. In addition, standard bolted connections, associated with connecting the panels to the rest of the building and making the panel to panel connections, are not ideal for preassembly.

Future research dealing with eliminating tolerance problems through the use of some type of self aligning connection, finding a way to use repetitive panels without the need for redundant members, and providing some type of boltless, possibly interlocking, connection to connect the panel to the building and make the connections between panels, might significantly increase the possibility of making preassembly of these types of systems successful in terms of both duration and safety.

Chapter 7: Conclusions

The objectives of this research are (1) to identify opportunities for the use of preassembly and prefabrication, (2) to develop ways to effectively determine the usefulness of these methods, and (3) to develop conceptual designs of innovative structures explicitly for preassembly to test the methodology.

The site observations of a project actually using preassembly methods provided data on the project impacts resulting from preassembly. Specifically, the method reduced the amount of worker time in the air by almost 50% and decreased the project duration by approximately 10%. Preassembly was used on a specific facility which consisted of three distinct structural frames. All of these frames used standard rolled steel sections with bolted connections, although two specific portions of the buildings used an innovative boltless beam-to-column connection (ATLSS Connection). While the building frame was designed specifically for preassembly, the configuration of the members was not systematically analyzed for its effect of the erection process.

From the site observations, we create a set of critical factors that reflects the interdependency of erection activities critical for preassembly of structural members. The critical factors are phase specific (from design through decking) and relate to the efficiency, capacity, performance, and safety of the activities performed for each phase. Within each factor, we include specific measures. For instance, in prefabrication efficiency, the complexity of the connection can be analyzed, including such specific counts as the number of holes to drill, the amount and type of welding required, and the number of pieces to attach. While these measures can be objectively determined and even quantified, we do not attempt to generalize these factors to all preassembly projects. Since construction projects are highly sensitive to design specifications and site conditions, a "preassembly" scale to fit all projects with the relative importance of the measures would be so generalized as to be useless. Instead, the set of critical factors could certainly be customized and scaled for a group of similar projects performed by a common team of actors (including designer, fabricator, and erector).

By explicitly considering the importance of design on the effectiveness of preassembly during erection, we focus attention on where changes can most easily be made to accommodate the new method. To complement these design activities, and to provide a basis for the development of innovative structural design concepts, we assemble a reference on standard components of structural systems using steel and precast concrete. In addition to a general reference on the standard connections in these materials, we also explore the logical relationships between the structural components as individual members and as preassembled sections.

The site observations, critical factors, and standard components provide a set of tools to identify opportunities for preassembly or prefabrication and to determine their usefulness. An additional method to objectively determine the impacts of the new methods is through a project impact analysis, which calculates the duration and safety effects of using the new methods for specific structural erection projects.

Building upon the insights gained from the development of the methods, we create innovative structural design concepts explicitly for preassembly. Although we generated many concepts for member configuration and alignment connections, this report includes those options which are currently the most feasible. We present three complete systems. The first system consists of single story tubular steel columns filled with concrete, with bay-size panels aligning within the tube during erection. The bay-size panels take advantage of the redundancy of the edge members by using small channels which, when combined, can carry the loads usually associated with larger single members. (We provide many options for this system.) System 2 consists of column trees, where beam stubs are rigidly preconnected to the column, with the bay-size panels which connect to the stubs at the point of moment inflection. In System 3, the most innovative and complex of the systems, the columns consist of either flared precast columns ("lily" columns) or collars inserted over the columns that provide an extended bearing surface for the panels. The panels themselves are bay-size slabs of concrete cast on the ground and then lifted by crane into position.

We analyze these three innovative systems in three different ways. First, we analyze the full set of options for each system and the system as a whole with respect to the critical factors to determine its performance during all project phases. Secondly, we structurally design and analyze the most promising options to evaluate their technical feasibility. Finally, using the insights gained from the critical factors and the specific member and connection designs from the structural analysis, we calculate the duration and safety impacts of the new systems. We compare these final project impacts to a building composed of standard structural rolled members erected using standard methods.

From these analyses, it appears that both Systems 1 and 2 are feasible and offer several advantages compared with standard methods. (Because of its technical complexity, the structural analysis of System 3 quickly grew beyond the scope of this report. Expert opinion is divided on the technical feasibility of this system.) However, the project impact analyses for both Systems 1 and 2 indicate that the use of innovative and nonstandard members and configurations significantly adds to the duration of the erection process, but does, even with the increased duration, reduce worker time in the air by 67% for System 1 and 31% for System 2.

Exploration of innovative structural systems highlights the interaction among the critical factors identified. While the innovative systems take advantage of the redundancy of the structural members by combining smaller members to function equivalently to single larger members, this approach does nonetheless add to the number of members which need to be arranged and connected, and therefore increases the project duration. In addition, the measurement and care taken to assure that the members are assembled correctly and will fit together when erected into their final locations requires great attention to the tolerances. The issue of tolerances remains one of the most significant issues that must be addressed for preassembly or prefabrication to compete as a viable alternative method. Finally, although we create concepts for innovative connections to align and permanently connect the preassembled sections, the technical complexity to develop these concepts and to demonstrate their feasibility places them beyond the range of this research. Therefore, the three systems generally use standard connections, although some of the alignment mechanisms are nonstandard. To fully exploit the potential advantages of preassembly and prefabrication, more research and design needs to be done on developing a family of connections specifically for the erection of large structural sections to completely eliminate the dangerous task of connection during erection.

This report presents the major findings from this research. The help of many members of the structural design, fabrication, and erection professions added significantly to the comprehensiveness and reliability of the results presented here. Since these professionals exhibited their willingness and interest in a systematic methodology to determine the opportunities and range of benefits and costs available from a new construction method, we hope that this report provides them with the insight and information that they need to continue with their exploration of this topic.

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Appendix B: Average and Interpolated Times of Activities

This Appendix lists the unit times for each activity, which we calculate from the actual data from the site observations. Each individual activity is isolated, and the times for activities that were not observed are interpolated or estimated. Times that are based on actual measured data are indicated with an underline and the number of measurements in parentheses. These production rates are used in the duration analysis of the DuPont buildings, discussed in Chapter 4. These isolated activities and production rates also form the basis for the project analyses of the new systems, discussed in Appendix E, although we make some modifications and add some activities which we did not observe.

UNLOADING OF ALL MEMBERS

We assume that it takes the same average time to unload each bundle of members, and we assume that each bundle contains the average number of members that we observed. The time for the workers and the crane to get into position (to the place where members are being unloaded) and for the truck to maneuver into position must also be considered, but these times are not estimated.

2.75 minutes/bundle (12) 1 bundle = 10 members (12)

SHAKEOUT OF ALL MEMBERS

We assume that it take to be the same average time to shakeout each bundle of the members.

9 minutes/bundle (1 bundle = 10 members)

ASSEMBLY OF PANELS/BENTS

Preparing area where panel/bent is to be assembled:

We assume that this depends on the size of the bent or panel to be assembled.

small panel/bent (<600 ft^2)	1 min
medium panel/bent (600-1200 ft^2)	3 min
large panel/bent (1200-2000 ft^2)	5 min
x-large panel/bent (>2000 ft^2)	7 min

Workers getting into position, getting bolts and tools:

5 minutes

Setting blocks in approximate locations before layout of members:

We assume that this depends on the size of the panel or bent being assembled. The complexity of the panel may also be important, but we do not quantify this.

small panel/bent (<600 ft^2)	5 min
medium panel/bent (600-1200 ft^2)	7 min
large panel/bent (1200-2000 ft^2)	10 min
x-large panel/bent (>2000 ft^2)	12 min

Getting crane into position, moving crane to first member:

3 minutes

Layout and attachment of each member:

We assume that this depends on the type and size of the member, how many places it is to be attached, and whether or not the position of the member must be measured.

Large beams/columns (>30 ft)

	0 conn.'s	1 conn.	2 conn.'s
Hooking	0.5 min	0.5 min	0.5 min
Lifting	1 min	1 min	1 min
Measuring[1]	3 min	2 min	-------
Positioning[1]	0.5 min	0.5 min	0.5 min
Adjusting blocks[1]	2 min	1 min	-------
Attaching[2]	-------	0.5 min	1 min
Unhooking	0.5 min	0.5 min	0.5 min
Moving to next member	1 min	1 min	1 min
Total elapsed time[3]	6 min (3)	5.25 min	4 min

Medium beams/columns (15-30 ft)

	0 conn.'s	1 conn.	2 conn.'s
Hooking	0.25 min	0.25 min	0.25 min
Lifting	0.75 min	0.75 min	0.75 min
Measuring[1]	2.5 min	1.5 min	-------
Positioning[1]	0.5 min	0.5 min	0.5 min
Adjusting blocks[1]	2 min	1 min	-------
Attaching[2]	-------	0.5 min	1 min
Unhooking	0.5 min	0.5 min	0.5 min
Moving to next member	1 min	1 min	1 min
Total elapsed time[3]	5 min	4.25 min	3.5 min (30)

Small beams/columns (<15 ft)

	0 conn.'s	1 conn.	2 conn.'s
Hooking	0.25 min	0.25 min	0.25 min
Lifting	0.5 min	0.5 min	0.5 min
Measuring[1]	2 min	1 min	-------
Positioning[1]	0.5 min	0.5 min	0.5 min
Adjusting blocks[1]	1.5 min	1 min	-------
Attaching[2]	-------	0.5 min	1 min
Unhooking	0.5 min	0.5 min	0.5 min
Moving to next member	1 min	1 min	1 min
Total elapsed time[3]	4.25 min	3.5 min	3.25 min

Other members

	Diagonal braces	Angles
Hooking	0.5 min	0.5 min
Lifting	0.75 min	0.75 min
Measuring[1]	-------	-------
Positioning[1]	0.5 min	3 min
Adjusting blocks[1]	-------	-------
Attaching[2]	1 min	2 min
Unhooking	0.5 min	0.5 min
Moving to next member	1 min	1 min
Total elapsed time[3]	3.75 min	6.75 min

Final measurement

This activity consists of measuring the diagonals and checking the levelness of the panel. We assume that this depends on the size of the panel or bent.

small panel/bent (<600 ft^2)	5 min
medium panel/bent (600-1200 ft^2)	7 min
large panel/bent (1200-2000 ft^2)	10 min
x-large panel/bent (>2000 ft^2)	12 min

Permanent connection

This depends on the number of bolts to install and tighten.

Installing remaining bolts (for one worker)

0.25 min/bolt

Tightening all bolts (for one worker)

3 min/bolt

ERECTION OF PANELS/BENTS

Attaching spreader bar to crane, moving crane with spreader bar to panel/bent:

This depends on whether the spreader bar is always attached and detached for each panel or bent.

6 minutes (1)

Workers getting into position, getting tools to workers:

12 minutes (workers in air) 5 minutes (workers on ground)

Attaching spreader bar to panel or bent, balancing

We assume that this depends on the size and weight of the section, and whether it is a panel or bent.

Bents

	light (<4 tons)	moderate (4-7 tons)	heavy (7-12 tons)	very heavy (>12 tons)
small (<600 ft^2)	2 min	2.5 min		
medium (600-1200 ft^2)	3 min	5 min	7 min	
large (1200-2000 ft^2)		10 min	12 min	
x-large (>2000 ft^2)			14 min	16 min

Panels

	light (<4 tons)	moderate (4-7 tons)	heavy (7-12 tons)	very heavy (>12 tons)
small (<600 ft^2)	2 min[4] (2)	4 min		
medium (600-1200 ft^2)	6 min	8 min	12 min	
large (1200-2000 ft^2)		14 min (1)	18 min	24 min[5] (1)
x-large (>2000 ft^2)			22 min	30 min

Lifting into position

We assume that this depends on the size, weight, and maneuverability of the section, and whether it is a panel or bent, although we did not quantify the maneuverability of the bent or panel, which depends on its destination and path it must take.

Bents

	light (<4 tons)	moderate (4-7 tons)	heavy (7-12 tons)	very heavy (>12 tons)
small (<600 ft^2)	3 min	3.5 min		
medium (600-1200 ft^2)	4 min	5 min	6 min	
large (1200-2000 ft^2)		7 min	8 min	10 min
x-large (>2000 ft^2)			10 min[4] (1)	12 min

Panels

	light (<4 tons)	moderate (4-7 tons)	heavy (7-12 tons)	very heavy (>12 tons)
small (<600 ft^2)	3 min[4] (2)	3.5 min		
medium (600-1200 ft^2)	4 min	5 min	8 min	
large (1200-2000 ft^2)		10 min (1)	14 min	18 min[5] (1)
x-large (>2000 ft^2)			18 min	25 min

Aligning panel/bent with connections

We assume that this depends on tolerances, which cannot exactly be taken account; therefore, we assume that the time for this activity is proportional to the size of the panel or bent.

	Bent	Panel
small (<600 ft^2)	3 min	2 min[4] (1)
medium (600-1200 ft^2)	5 min	10 min
large (1200-2000 ft^2)	7 min	15 min
x-large (>2000 ft^2)	10 min[4] (1)	18 min

Connecting panel/bent to building

This depends on the number of connections that are made to the building.

0.5 min/bolt installed (for one worker)
4 min/ATLSS connector installed (for one worker)

Unhooking panel/bent

We assume that this depends on whether the section is a bent or panel and the size of panel).

bent (any size)	1 min (1)
small panel (<600 ft^2)	3.5 min (2)
medium panel (600-1200 ft^2)	6 min
large panel (1200-2000 ft^2)	8 min (1)
x-large panel (>2000 ft^2)	10 min

Moving crane back to the ground, detaching spreader bar:

8 minutes (1)

Workers climbing down:

5 minutes (workers in air)

INSTALLING GUY WIRES TO SUPPORT BENT

9 min/bent (1)

ERECTION OF STICK-BUILT MEMBERS

Workers getting into position, getting bolts and tools:

15 minutes

Getting crane into position, moving crane to first member:

3 minutes

Lifting and attachment of each member

We assume that this depends on the type and size of the member being erected.

	Large beams (>30 ft)	Medium beams (15-30 ft)	Small beams (<15 ft)
Hooking	0.5 min	0.5 min	0.5 min
Lifting	1 min	0.75 min	0.75 min
Positioning/aligning	1.5 min	1.25 min	1 min
Attaching	2 min	2 min[2]	2 min[2]
Unhooking	1 min	0.75 min	0.5 min
Moving to next member	1 min	1 min	1 min
Total elapsed time[3]	6 min	5.25 min	4.75 min

	Columns	Diagonal braces	Angles
Hooking	1 min	0.75 min	0.75 min
Lifting	1 min	0.75 min	0.75 min
Positioning/aligning	3 min	2 min	2 min
Attaching	2 min[2]	2 min[2]	4 min[2]
Unhooking	1 min	0.5 min	0.5 min
Moving to next member	1 min	1 min	1 min
Total elapsed time[3]	8 min	5 min	7 min

Appendix C: Descriptions of Critical Factor Measures

This Appendix describes the different measures of the critical factors that we identify, which we discussed in section 4.2. The objective of the measures is to indicate whether certain aspects of the project, such as the size of a preassembled panel, have a positive or negative impact on the critical factors. For example, the efficiency of erection increases as the number of units to be erected is reduced; however, as the sizes of the units increase (e.g., panels with several members assembled on the ground as opposed to individual members), the erection efficiency decreases. In this way, tradeoffs between the critical factors and measures can be identified.

Design Efficiency:

1) *Standardization of structural elements*--design efficiency decreases with use of nonstandard structural elements.
2) *Standardization of structural configuration*--design efficiency decreases with use of nonstandard structural configurations.
3) *Repetition of design units*--design efficiency increases with increased repetition of design units.
4) *Member redundancy*--design efficiency increases as member redundancy (number of extra members to design) decreases.

Fabrication Efficiency:

1) *Connection complexity*--fabrication efficiency increases with the decrease in connection complexity, which depends on the following factors:

    a) *Number of holes to drill*--connection complexity decreases as the number of holes to drill for each connection decreases.
    b) *Amount of welding*--connection complexity decreases as the amount of welding necessary for each connection decreases.
    c) *Type of welding*--connection complexity decreases with the use of simpler welds.
    d) *Number of pieces to attach*--connection complexity decreases as the number of pieces (plates, angles, tees) to attach to the member decreases.

2) *Number of connections per member*--fabrication efficiency increases as the number of connections per member decreases.
3) *Member complexity*--fabrication efficiency increases as member complexity decreases, which depends on the following factors:

    a) *Topology (number of faces)*--member complexity decreases as the number of faces that need to be worked on for each member decreases.
    b) *Congestion in working surface*--member complexity decreases with the decrease of congestion in the working surface of the members.

4) *Number of members*--fabrication efficiency increases as the number of members to be fabricated decreases.
5) *Tolerances*--fabrication efficiency increases as the need for strict tolerances decreases.
6) *Resources*--fabrication efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

    a) *Production rates*--fabrication efficiency increases as production rates of the resources increase.
    b) *Amount and type of resources*--fabrication efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).
    c) *Availability of resources*--fabrication efficiency increases as the delay in availability of qualified resources decreases.
    d) *Flexibility of resources*--fabrication efficiency increases with the increase in flexibility of resources.
    e) *Utilization*--fabrication efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

7) *Special equipment requirements*--fabrication efficiency increases with the decrease in special equipment requirements, which are not readily available.
8) *Special activity requirements*--fabrication efficiency increases with the decrease in special activity requirements.

Fabrication Capacity:

1) *Equipment capacity*--fabrication capacity increases as the capacity of the erection equipment increases, which depends on the following factors:

  a) *Equipment load capacity*--equipment capacity increases as the load capacity of the equipment increases.

  b) *Production rates*--equipment capacity increases as the production rates of the equipment increase.

2) *Shop layout*--fabrication capacity depends upon the layout of the shop and the storage area available.

3) *Degree of automation*--fabrication capacity increases as the degree of automation increases, up to a maximum level.

Fabrication Safety:

1) *Labor hours*--fabrication safety increases as labor hours required decreases.

2) *Danger exposure*--fabrication safety increases as danger exposure of the workers decreases.

3) *Special personal equipment*--fabrication safety increases with the use of special personal equipment.

Prefabrication Efficiency:

1) *Connection complexity*--prefabrication efficiency increases with the decrease in connection complexity, which depends on the following factors:

  a) *Number of holes to drill*--connection complexity decreases as the number of holes to drill for each connection decreases.

  b) *Amount of welding*--connection complexity decreases as the amount of welding necessary for each connection decreases.

  c) *Type of welding*--connection complexity decreases with the use of simpler welds.

  d) *Number of pieces to attach*--connection complexity decreases as the number of pieces (plates, angles, tees) to attach decreases.

  e) *Number of bolts to install*--connection complexity decreases as the number of bolts to install for each connection decreases.

  f) *Number of bolts to tighten*--connection complexity decreases as the number of bolts to tighten for each connection decreases.

  g) *Necessary degree of torque*--connection complexity decreases as the necessary degree of torque for the bolts decreases (e.g., snug-tight versus slip critical).

2) *Number of connections per member*--prefabrication efficiency increases as the number of connections per member decreases.

3) *Size of members*--prefabrication efficiency increases as the sizes of the members in the units decrease to a base size; below this base size, the prefabrication efficiency decreases.

4) *Weight of members*--prefabrication efficiency increases as the weights of the members in the units decrease.

5) *Member complexity*--prefabrication efficiency increases as member complexity decreases, which depends on the following factors:

  a) *Topology (number of faces)*--member complexity decreases as the number of faces that need to be worked on for each member decreases.

  b) *Congestion in working surface*--member complexity decreases with the decrease of congestion in the working surface of the members.

6) *Ease of member placement*--prefabrication efficiency increases as the ease of member placement increases.

7) *Measurement*--prefabrication efficiency increases as the amount of necessary measurement for each member with respect to the other members in the unit decreases.

8) *Number of members per unit*--prefabrication efficiency increases as the number of members per unit decreases.

9) *Number of units*--prefabrication efficiency increases with the decrease in the number of units to be prefabricated.

10) *Tolerances*--prefabrication efficiency increases as the need for strict tolerances decreases.

11) *Resources*--prefabrication efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

  a) *Production rates*--prefabrication efficiency increases as production rates of the resources increase.

  b) *Amount and type of resources*--prefabrication efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).

c) *Availability of resources*--prefabrication efficiency increases as the delay in availability of qualified resources decreases.

d) *Flexibility of resources*--prefabrication efficiency increases with the increase in flexibility of resources.

e) *Utilization*--prefabrication efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

12) *Special equipment requirements*--prefabrication efficiency increases with the decrease in special equipment requirements, which are not readily available.

13) *Special activity requirements*--prefabrication efficiency increases with the decrease in special activity requirements.

14) *Special site constraints*--prefabrication efficiency increases with the decrease in special site constraints.

Prefabrication Capacity:

1) *Equipment capacity*--prefabrication capacity increases as the capacity of the erection equipment increases, which depends on the following factors:

a) *Equipment load capacity*--equipment capacity increases as the equipment load capacity increases.

b) *Angle*--equipment capacity increases as the angle of lift (to the vertical) decreases, within the capacity of the operation equipment.

c) *Swing*--equipment capacity increases as the allowable length of swing increases, within the needed load range.

2) *Laydown area (space)*--prefabrication capacity increases as the available laydown area increases, up to the maximum area needed.

Prefabrication Performance:

1) *Stability*--prefabrication performance increases as the prefabrication stability increases, which depends on the following factors:

a) *Stability of each member*--prefabrication stability increases as the stability of each member increases (e.g. rest on jig or template).

2) *Durability*--prefabrication performance increases as the durability of the members increases (w.r.t. impacts on the members during prefabrication).

3) *Capacity*--prefabrication performance increases as the capacity of the members increases (w.r.t. prefabrication loads).

Prefabrication Safety:

1) *Labor hours*--prefabrication safety increases as labor hours required decreases.

2) *Danger exposure*--prefabrication safety increases as danger exposure of the workers decreases.

3) *Special personal equipment*--prefabrication safety increases with the use of special personal equipment.

Transportation Efficiency:

1) *Number of units*--transportation efficiency increases as the number of units to transport decreases, although there is an optimum range, depending on the sizes and weights of the units and the method of transportation.

2) *Size of units*--transportation efficiency increases as the sizes of the units decrease, although there is an optimum range, depending on the number of units and weights of the units and the method of transportation.

3) *Weight of units*--transportation efficiency increases as the weights of the units decrease, although there is an optimum range, depending on the number of units and sizes of the units and the method of transportation.

4) *Density of units*--transportation efficiency increases as the densities of the units increase (e.g., when there is little open space within the unit).

5) *Ease of stacking*--transportation efficiency increases as the ease of stacking increases.

6) *Resources*--transportation efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

a) *Production rates*--transportation efficiency increases as production rates of the resources increase.

b) *Amount and type of resources*--transportation efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).

c) *Availability of resources*--transportation efficiency increases as the delay in availability of qualified resources decreases.

d) *Flexibility of resources*--transportation efficiency increases with the increase in flexibility of resources.

e) *Utilization*--transportation efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

7) *Special equipment requirements*--transportation efficiency increases with the decrease in special equipment requirements, which are not readily available.

8) *Special activity requirements*--transportation efficiency increases with the decrease in special activity requirements.

9) *Special site constraints*--transportation efficiency increases with the decrease in special site constraints (e.g. road access).

Transportation Capacity:

1) *Equipment capacity*--transportation capacity increases as the capacity of the transportation equipment increases, which depends on the following factors:

a) *Equipment load capacity*--equipment capacity increases as the load capacity of the equipment increases.

b) *Volumetric capacity*--equipment capacity increases as the volumetric capacity of the transportation unit increases.

Transportation Performance:

1) *Stability*--transportation performance increases as the transportation stability increases, which depends on the following factors:

a) *Center of gravity*--transportation stability increases as the center of gravity of the unit approaches the center of gravity of the transportation unit.

2) *Durability*--transportation performance increases as the durability of the members increases (w.r.t. impacts on the members during transportation).

3) *Capacity*--transportation performance increases as the capacity of the members increases (w.r.t. transportation loads).

Transportation Safety:

1) *Labor hours*--transportation safety increases as labor hours required decreases.

2) *Danger exposure*--transportation safety increases as danger exposure of the workers decreases.

3) *Special personal equipment*--transportation safety increases with the use of special personal equipment.

Handling Efficiency:

1) *Number of units*--handling efficiency increases as the number of units to handle decreases, although there is an optimum range, depending on the sizes and weights of the units and the handling equipment.

2) *Size of units*--handling efficiency increases as the sizes of the units decrease, although there is an optimum range, depending on the number of units and weights of the units and the handling equipment.

3) *Weight of units*--handling efficiency increases as the weights of the units decrease, although there is an optimum range, depending on the number of units and sizes of the units and the handling equipment.

4) *Ease of stacking*--handling efficiency increases as the ease of stacking increases.

5) *Resources*--handling efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

a) *Production rates*--handling efficiency increases as production rates of the resources increase.

b) *Amount and type of resources*--handling efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).

c) *Availability of resources*--handling efficiency increases as the delay in availability of qualified resources decreases.

d) *Flexibility of resources*--handling efficiency increases with the increase in flexibility of resources.

e) *Utilization*--handling efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

6) *Special equipment requirements*--handling efficiency increases with the decrease in special equipment requirements, which are not readily available.

7) *Special activity requirements*--handling efficiency increases with the decrease in special activity requirements.
8) *Special site constraints*--handling efficiency increases with the decrease in special site constraints.

Handling Capacity:

1) *Equipment capacity*--handling capacity increases as the capacity of the handling equipment increases, which depends on the following factors:

a) *Equipment load capacity*--equipment capacity increases as the load capacity of the handling equipment increases.

b) *Angle*--equipment capacity increases as the angle of lift (to the vertical) decreases, within the capacity of the operation equipment.

c) *Swing*--equipment capacity increases as the allowable length of swing increases, within the needed load range.

2) *Laydown area (space)*--handling capacity increases as the laydown area for unloading and shakeout increases, up to the maximum area needed.

Handling Performance:

1) *Stability*--handling performance increases as the handling stability increases, which depends on the following factors:

a) *Center of gravity*--handling stability increases as the center of gravity of the unit approaches the center of gravity of the handling unit.

2) *Durability*--handling performance increases as the durability of the members increases (w.r.t. impacts on the members during handling).
3) *Capacity*--handling performance increases as the capacity of the members increases (w.r.t. handling loads).

Handling Safety:

1) *Labor hours*--handling safety increases as labor hours required decreases.
2) *Danger exposure*--handling safety increases as danger exposure of the workers decreases.
3) *Special personal equipment*--handling safety increases with the use of special personal equipment.

Preassembly Efficiency:

1) *Connection complexity*--preassembly efficiency increases with the decrease in connection complexity, which depends on the following factors:

a) *Number of holes to drill*--connection complexity decreases as the number of holes to drill for each connection decreases.

b) *Amount of welding*--connection complexity decreases as the amount of welding necessary for each connection decreases.

c) *Type of welding*--connection complexity decreases with the use of simpler welds.

d) *Number of pieces to attach*--connection complexity decreases as the number of pieces (plates, angles, tees) to attach decreases.

e) *Number of bolts to install*--connection complexity decreases as the number of bolts to install for each connection decreases.

f) *Number of bolts to tighten*--connection complexity decreases as the number of bolts to tighten for each connection decreases.

g) *Necessary degree of torque*--connection complexity decreases as the necessary degree of torque for the bolts decreases (e.g., snug-tight versus slip critical).

2) *Number of connections per member*--preassembly efficiency increases as the number of connections per member decreases.
3) *Size of members*--preassembly efficiency increases as the sizes of the members decrease.
4) *Weight of members*--preassembly efficiency increases as the weights of the members decrease.
5) *Member complexity*--preassembly efficiency increases as member complexity decreases, which depends on the following factors:

a) *Topology (number of faces)*--member complexity decreases as the number of faces that need to be worked on for each member decreases.

b) *Congestion in working surface*--member complexity decreases with the decrease of congestion in the working surfaces of the members.

6) *Ease of member placement*--preassembly efficiency increases as the ease of member placement increases.

7) *Measurement*--preassembly efficiency increases as the amount of necessary measurement with respect to the other members in the unit decreases.

8) *Number of members per unit*--preassembly efficiency increases as the number of members per unit to be preassembled decreases.

9) *Number of units*--preassembly efficiency increases as the number of units to be preassembled decreases.

10) *Tolerances*--preassembly efficiency increases as the need for strict tolerances decreases.

11) *Resources*--preassembly efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

a) *Production rates*--preassembly efficiency increases as production rates of the resources increase.

b) *Amount and type of resources*--preassembly efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).

c) *Availability of resources*--preassembly efficiency increases as the delay in availability of qualified resources decreases.

d) *Flexibility of resources*--preassembly efficiency increases with the increase in flexibility of resources.

e) *Utilization*--preassembly efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

12) *Special equipment requirements*--preassembly efficiency increases with the decrease in special equipment requirements, which are not readily available.

13) *Special activity requirements*--preassembly efficiency increases with the decrease in special activity requirements.

14) *Special site constraints*--preassembly efficiency increases as the number of special site constraints decreases.

Preassembly Capacity:

1) *Equipment capacity*--preassembly capacity increases as the capacity of the preassembly equipment increases, which depends on the following factors:

a) *Equipment load capacity*--equipment capacity increases as the load capacity of the erection equipment increases.

b) *Angle*--equipment capacity increases as the angle of lift (to the vertical) decreases, within the capacity of the operation equipment.

c) *Swing*--equipment capacity increases as the allowable length of swing increases, within the needed load range.

2) *Laydown area (space)*--preassembly capacity increases as the available laydown area increases, up to the maximum area needed.

Preassembly Performance:

1) *Stability*--preassembly performance increases as the preassembly stability increases, which depends on the following factors:

a) *Stability of each member*--preassembly stability increases as the stability of each member increases (e.g. rest on jig or template).

2) *Durability*--preassembly performance increases as the durability of the members increases (w.r.t. impacts on the members during preassembly).

3) *Capacity*--preassembly performance increases as the capacity of the members increases (w.r.t. preassembly loads).

Preassembly Safety:

1) *Labor hours*--preassembly safety increases as labor hours required decrease.

2) *Danger exposure*--preassembly safety increases as danger exposure of the workers decreases.

3) *Special personal equipment*--preassembly safety increases with the use of special personal equipment.

Erection Efficiency:

1) *Connection complexity*--erection efficiency increases with the decrease in connection complexity, which depends on the following factors:

a) *Number of holes to drill*--connection complexity decreases as the number of holes to drill for each connection increases.

b) *Amount of welding*--connection complexity decreases as the amount of welding necessary for each connection increases.

c) *Type of welding*--connection complexity decreases with the use of simpler welds.

d) *Number of pieces to attach*--connection complexity decreases as the number of pieces (plates, angles, tees) to attach decreases.

e) *Number of bolts to install*--connection complexity decreases as the number of bolts to install for each connection decreases.

f) *Number of bolts to tighten*--connection complexity decreases as the number of bolts to tighten for each connection decreases.

g) *Necessary degree of torque*--connection complexity decreases as the necessary degree of torque for the bolts decreases (e.g., snug-tight versus slip critical).

2) *Number of connections per unit*--erection efficiency increases as the number of connections per unit decreases.
3) *Size of units*--erection efficiency increases as the sizes of the units decrease.
4) *Weight of units*--erection efficiency increases as the weights of the units decrease.
5) *Unit complexity*--erection efficiency increases as unit complexity decreases, which depends on the following factors:

a) *Topology of connections*--unit complexity decreases as the number of faces that need to be worked on during connection of each unit decreases.

b) *Congestion in working surface*--unit complexity decreases with the decrease of congestion in the working surfaces of the units.

6) *Ease of unit placement*--erection efficiency increases as the ease of unit placement increases.
7) *Number of units*--erection efficiency increases as the number of units to erect decreases.
8) *Tolerances*--erection efficiency increases as the tolerances get tighter, and as the need for strict tolerances decreases.
9) *Height*--erection efficiency increases as the working height decreases.
10) *Resources*--erection efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

a) *Production rates*--erection efficiency increases as production rates of the resources increase.

b) *Amount and type of resources*--erection efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).

c) *Availability of resources*--erection efficiency increases as the delay in availability of qualified resources decreases.

d) *Flexibility of resources*--erection efficiency increases with the increase in flexibility of resources.

e) *Utilization*--erection efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

11) *Special equipment requirements*--erection efficiency increases with the decrease in special equipment requirements, which are not readily available.
12) *Special activity requirements*--erection efficiency increases with the decrease in special activity requirements (e.g., balancing panels).
13) *Special site constraints*--erection efficiency increases as the number of special site constraints decreases.

Erection Capacity:

1) Equipment capacity--erection capacity increases as the capacity of the erection equipment increases, which depends on the following factors:

a) *Equipment load capacity*--equipment capacity increases as the load capacity of the erection equipment increases.

b) *Angle*--equipment capacity increases as the angle of lift (to the vertical) decreases, within the capacity of the operation equipment.

c) *Swing*--equipment capacity increases as the allowable length of swing increases, within the needed load range.

Erection Performance:

1) *Stability*--erection performance increases as the erection stability increases, which depends on the following factors:

a) *Center of gravity*--erection stability increases as the center of gravity of the unit approaches the center of the lifting forces of the erection equipment.

b) *Sequence*--erection stability depends on the order in which units are erected.

c) *Erected position with respect to previously erected units*--erection stability depends on the orientation of the unit:

Horizontal--erection stability increases as the center of gravity of the unit after setting approaches the center of gravity of the adjacent previously erected units.

Vertical--erection stability increases as the moment arm of the center of gravity of the unit after setting with respect to the base decreases.

d) *Connection proportion complete*--erection stability increases with the increase in the proportion of connections completed during erection.

2) *Durability*--erection performance increases as the durability of the members increases (w.r.t. impacts on the members during erection).

3) *Capacity*--erection performance increases as the capacity of the members increases (w.r.t. erection loads).

Erection Safety:

1) *Labor hours*--erection safety increases as labor hours required (especially in the air) decrease.

2) *Danger exposure*--erection safety increases as danger exposure of the workers decreases.

3) *Special personal equipment*--erection safety increases with the use of special personal equipment.

Plumbing Efficiency:

1) *Number of units to plumb*--plumbing efficiency increases as the number of units to plumb decreases.

2) *Ease of plumbing*--plumbing efficiency increases as the ease of adjusting unplumbed sections increases (e.g. less rigid structure).

3) *Tolerances*--plumbing efficiency increases as the acceptable range for tolerance requirements decreases.

4) *Resources*--plumbing efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

a) *Production rates*--plumbing efficiency increases as production rates of the resources increase.

b) *Amount and type of resources*--plumbing efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).

c) *Availability of resources*--plumbing efficiency increases as the delay in availability of qualified resources decreases.

d) *Flexibility of resources*--plumbing efficiency increases with the increase in flexibility of resources.

e) *Utilization*--plumbing efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

5) *Special equipment requirements*--plumbing efficiency increases with the decrease in special equipment requirements, which are not readily available.

6) *Special activity requirements*--plumbing efficiency increases with the decrease in special activity requirements.

Plumbing Safety:

1) *Labor hours*--plumbing safety increases as labor hours required (especially in the air) decrease.

2) *Danger exposure*--plumbing safety increases as danger exposure of the workers decreases.

3) *Special personal equipment*--plumbing safety increases with the use of special personal equipment.

Permanent Connection Efficiency:

1) *Connection complexity*--permanent connection efficiency increases with the decrease in connection complexity, which depends on the following factors:

a) *Number of holes to drill*--connection complexity decreases as the number of holes to drill for each connection increases.

b) *Amount of welding*--connection complexity decreases as the amount of welding necessary for each connection increases.

c) *Type of welding*--connection complexity decreases with the use of simpler welds.

d) *Number of pieces to attach*--connection complexity decreases as the number of pieces (plates, angles, tees) to attach decreases.

e) *Number of bolts to install*--connection complexity increases as the number of bolts to install for each connection decreases.

f) *Number of bolts to tighten*--connection complexity decreases as the number of bolts to tighten for each connection decreases.

g) *Necessary degree of torque*--connection complexity decreases as the necessary degree of torque for the bolts decreases (e.g., snug-tight versus slip critical).

2) *Number of connections per unit*--permanent connection efficiency increases as the number of connections per unit decreases.

3) *Unit complexity*--permanent connection efficiency increases as unit complexity decreases, which depends on the following factors:

a) *Topology of connections*--unit complexity decreases as the number of faces that need to be worked on during connection of each unit decreases.

b) *Congestion in working surface*--unit complexity decreases with the decrease of congestion in the working surfaces of the pieces.

4) *Number of units*--permanent connection efficiency increases with the decrease in the number of units to be erected.

5) *Height*--permanent connection efficiency increases as the working height decreases.

6) *Resources*--permanent connection efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

a) *Production rates*--permanent connection efficiency increases as production rates of the resources increase.

b) *Amount and type of resources*--permanent connection efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).

c) *Availability of resources*--permanent connection efficiency increases as the delay in availability of qualified resources decreases.

d) *Flexibility of resources*--permanent connection efficiency increases with the increase in flexibility of resources.

e) *Utilization*--permanent connection efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

7) *Special equipment requirements*--permanent connection efficiency increases with the decrease in special equipment requirements, which are not readily available.

8) *Special activity requirements*--permanent connection efficiency increases with the decrease in special activity requirements (e.g., grouting).

Permanent Connection Safety:

1) *Labor hours*--permanent connection safety increases as labor hours required (especially in the air) decrease.

2) *Danger exposure*--permanent connection safety increases as the danger exposure of workers decreases.

3) *Special personal equipment*--permanent connection safety increases with the use of special personal equipment.

Decking/Slab Efficiency:

1) *Floor complexity*--decking/slab efficiency increases with the decrease in the floor complexity, which depends on the following:

a) *Complexity of layout*--floor complexity decreases with the decrease in complexity of the layout (e.g. rectangular).

b) *Number of sheets to install*--floor complexity decreases with the decrease in the number of decking sheets to install.

c) *Number of shear studs to install*--floor complexity decreases with the decrease in the number of shear studs to install.

d) *Amount of other components to install*--floor complexity decreases with the decrease in the amount of other components to install (e.g. rebar).

e) *Slab depth*--floor complexity decreases with the decrease in depth of slab to pour.

f) *Slab area*--floor complexity decreases with the decrease in slab area to pour.

2) *Number of floors*--decking/slab efficiency increases as the number of floors decreases, to a minimum practical number.

3) *Congestion of floor*--decking/slab efficiency increases with decreased floor congestion.

4) *Resources*--decking/slab efficiency depends on the resources (e.g., specialized workers, equipment) in the following ways:

a) *Production rates*--decking/slab efficiency increases as production rates of the resources increase.

b) *Amount and type of resources*--decking/slab efficiency increases as the amount and type of available resources increases, up to a maximum level (e.g., special equipment, workers with the right kind of experience, knowledge, and work ethic).

c) *Availability of resources*--decking/slab efficiency increases as the delay in availability of qualified resources decreases.

d) *Flexibility of resources*--decking/slab efficiency increases with the increase in flexibility of resources.

e) *Utilization*--decking/slab efficiency increases with the increase in the proportion of time the resources are being used, up to a maximum level.

5) *Special equipment requirements*--decking/slab efficiency increases with the decrease in special equipment requirements, which are not readily available.

6) *Special activity requirements*--decking/slab efficiency increases with the decrease in special activity requirements.

7) *Special site constraints*--decking/slab efficiency increases as the number of special site constraints decreases.

Decking/Slab Capacity:

1) *Equipment capacity*--decking/slab capacity increases as the capacity of the decking/slab equipment increases, which depends on the following factors:

a) *Volumetric capacity (pump/bucket/chute)*--equipment capacity increases as the volumetric capacity of the equipment increases.

b) *Height*--equipment capacity increases as the height decreases, depending on the capacity of the equipment.

Decking/Slab Performance:

1) *Stability*--decking/slab performance increases as the stability of the noncomposite structure increases.

2) *Capacity*--decking/slab performance increases as the capacity of the noncomposite structure increases.

Decking/Slab Safety:

1) *Labor hours*--decking/slab safety increases as labor hours required (especially in the air) decrease.

2) *Danger exposure*--decking/slab safety increases as the danger exposure of the workers decreases.

3) *Special personal equipment*--decking/slab safety increases with the use of special personal equipment.

Appendix D: Structural Analysis Calculations

This section describes the calculations we use to perform a basic feasibility check of the new systems described in Chapter 5, and to get the approximate required dimensions of members. We design the systems with respect to *gravity loads only*, assuming that an appropriate means of resisting lateral loads (e.g., wind and seismic loads), such as bracing members or additional reinforcement of connections, would be provided.

PROTOTYPE BUILDING

To provide a basis for comparison between the new systems and a traditionally constructed system in the project analyses, and to provide a means of checking the feasibility of the new systems, we develop the prototype building shown in Figure D-1. This prototype building is five stories high and has five 30 *ft* bays in the longitudinal direction and four 25 *ft* bays in the transverse direction. We perform a structural performance and project duration analysis for the prototype building using each of the systems and compare the results. The live and dead loads applied to the prototype building are those for a typical office building.

CALCULATION OF LOADS

Calculating the dead loads:

We calculate the dead loads assuming typical structural materials and services will be used. If the components of a particular system differ significantly from these assumptions, the dead loads must be recalculated and checked. Table D-1 lists the individual and total dead loads for the roof and other floors.

Table D-1: Dead loads.

Roof dead loads		**Other floor dead loads**	
Ballast	15 *psf*	Concrete slabs (3 *in*)	40 *psf*
Insulation	2 *psf*	Structural steel	10 *psf*
Structural steel	10 *psf*	Mechanical system	5 *psf*
Mechanical system	5 *psf*	Electrical system	2 *psf*
Electrical system	2 *psf*	Sprinkler system	3 *psf*
Sprinkler system	3 *psf*	Ceiling	2 *psf*
Ceiling	2 *psf*		
Total	39 *psf* → **40 *psf***	**Total**	62 *psf* → **65 *psf***

Calculating the live loads:

The live loads we use are those of a typical office building, 50 *psf*.

Summary of loads:

Table D-2 summarizes the gravity loads we use in the structural analysis calculations.

Table D-2: Gravity loads.

Live loads		**Dead loads**	
All floors:	50 *psf*	All floors:	65 *psf*
Roof:	50 *psf*	Roof:	40 *psf*

Load cases:

We use the Load and Resistance Factor Design Method (LRFD) to perform the structural analyses, with the two load cases that involve only gravity loads, shown in Table D-3.

Table D-3: Load cases involving only gravity loads.

Load case 1: $1.4\ D$ Load case 2: $1.2\ D + 1.6\ L + 0.5\ L_r$	D = dead loads L = floor live loads L_r = roof live loads

STRUCTURAL ANALYSIS OF SYSTEM 1

COLUMNS

Finding the required capacity of the columns:

First, we determine the required axial capacity for an interior column on the first floor (subjected to gravity loads only). It should be noted that moments will be introduced into the columns for nonuniform live load distributions and exterior columns. However, since our purpose is simply to get a very rough estimate of the column sizes, we neglect these moments and only design for the axial force due to the gravity loads. A larger column will likely be required to withstand these moments, in addition to the axial loads, but with the wide range of available columns of this type, procuring a larger column to resist these moments should not limit the feasibility of this system. Table D-2 lists the gravity loads we use in these calculations.

To find the axial force in a first-story interior column (worst case), for each of the load cases listed in Table D-3, we sum the forces acting on the tributary areas of the columns for all of the floors above the first story (with the appropriate load factors), and we choose the highest load.

Load case 1: $1.4\ D$

$$P_u = 1.4[40\ psf + 4(65\ psf)](25\ ft)(30\ ft)$$
$$P_u = 315{,}000\ lb = 315\ k$$

Load case 2: $1.2\ D + 1.6\ L + 0.5\ L_r$

$$P_u = \{1.2[40\ psf + 4(65\ psf)] + 1.6[4(50\ psf)] + 0.5(50\ psf)\}(25\ ft)(30\ ft)$$
$$P_u = 528{,}750\ lb = 529\ k$$

Maximum required axial capacity of columns:

$\mathbf{P_u = 529\ k}$ (from load case 2)

Designing the composite columns:

After finding the maximum required axial capacity of a first-story interior column, we refer to the LRFD manual to choose a composite column.

Assumptions:

$P_u = 529\ k$
$K = 1.2$ (assuming rotation fixed, translation free)
$L = 12\ ft$ (story height)
$KL = (1.2)(12\ ft) = 14.4\ ft$ (effective length)

From LRFD Manual:

Choose 12x8, 3/8" thick concrete filled tube.
$\phi P_n = 615\ k$ (assuming $f'_c = 3.5\ ksi$ and $F_y = 46\ ksi$)
$weight = 47.90\ lb/ft$

GIRDERS

Finding the required capacity of the girders before the concrete cures (dead loads only):

Because the girders do not act compositely until the concrete cures, it is necessary to find the required capacity of the noncomposite steel section subjected to dead loads. We assume that the beams are simply supported (worst case). Since this is a one-way system, we have a choice as to whether the channel girders run in the short direction of the prototype building (Frame A) or in the long direction (Frame B); see Figure D-2. We check the girders in an interior frame for each of these cases (refer to Table D-2 for the gravity loads we use).

For Frame A: (30 *ft* between frames)

$\omega_{DL} = (65\ psf)(30\ ft) = 1950\ lb/ft = 1.95\ k/ft$ (for floors other than roof)
$\omega_u = 1.4\ \omega_{DL} = 1.4(1.95\ k/ft) = 2.73\ k/ft$
$L = 25\ ft$

$$M_u^+ = \frac{\omega_u L^2}{8} = \frac{(2.73\ k/ft)(25\ ft)^2}{8} = \mathbf{213\ k\text{-}ft}$$

$$V_u = \frac{\omega_u L}{2} = \frac{(2.73\ k/ft)(25\ ft)}{2} = \mathbf{34.1\ k}$$

For Frame B: (25 *ft* between frames)

$\omega_{DL} = (65\ psf)(25\ ft) = 1625\ lb/ft = 1.625\ k/ft$ (for floors other than roof)
$\omega_u = 1.4\ \omega_{DL} = 1.4(1.625\ k/ft) = 2.275\ k/ft$
$L = 30\ ft$

$$M_u^+ = \frac{\omega_u L^2}{8} = \frac{(2.275\ k/ft)(30\ ft)^2}{8} = \mathbf{256\ k\text{-}ft}$$

$$V_u = \frac{\omega_u L}{2} = \frac{(2.275\ k/ft)(30\ ft)}{2} = \mathbf{34.1\ k}$$

Designing the noncomposite girders (dead loads only):

After finding the required moment and shear capacity of a noncomposite channel girder for each frame option, we need to choose an appropriate section. Figures D-3 and D-4 show the two configuration options which we will consider; Option 1 is made up of two channels in a box-beam configuration with a bottom plate (see Figure D-3), and Option 2 includes two channels in an I-beam configuration without a bottom plate (see Figure D-4). To choose a section for each option, we calculate the moment and shear capacities of noncomposite sections with various channel sections and plate thicknesses, and choose one with sufficient capacity. We calculate the moment capacities of the sections based on first yielding of the exterior fibers of the section, assuming that the spanning elements provide enough lateral stability to avoid lateral torsional buckling. We calculate the shear capacities of the sections by adding the shear capacities of the component channels (neglecting any contributions from the plate). We show two examples of these calculations, corresponding to the two configuration options. In Example 1, which is applicable to girder configuration option 1, we show the calculation of the capacity of a section with C15x50 channels and a 0.5 *in* thick plate; Table D-4 shows the results for other channel sizes and plate thicknesses, which we calculate using a spreadsheet. Then, we choose a section from Table D-4 that satisfies the moment and shear requirements. In Example 2, which is applicable to girder configuration option 2, we use a similar procedure, calculating the capacity of a section with C15x50 channels and no bottom plate; Table D-5 shows these results for other channel sizes. Then, we choose a section from Table D-5 that satisfies the moment and shear requirements. The sections chosen to satisfy these noncomposite strength requirements must, of course, also have enough strength when the section acts compositely (when the concrete cures) with the full load application (dead and live loads); this will be checked in the next section.

Example 1: Girder configuration option 1

Properties of one channel (C15x50):

$A = 14.7\ in^2$	(area)
$d = 15.00\ in$	(depth)
$b_f = 3.716\ in$	(flange width)
$I_x = 404\ in^4$	(moment of inertia with respect to x-axis)
$\phi V_n = 209\ k$	(shear capacity, from LRFD manual)

Properties of plate:

$w_p = 2b_f = 2(3.716\ in) = 7.432\ in$	(width)
$t_p = 0.5\ in$	(thickness)
$A = (7.432\ in)(0.5\ in) = 3.716\ in^2$	

Properties of entire section:

$A = 2(14.7\ in^2) + 3.716\ in^2 = 33.1\ in^2$

y_c = distance to the neutral axis from the bottom of the section

$$y_c = \frac{2(14.7\ in^2)\left(\dfrac{15.00\ in}{2} + 0.5\ in\right) + (3.716\ in^2)(0.25\ in)}{33.1\ in^2}$$

$y_c = 7.13\ in$

$$I_x = 2(404\ in^4) + 2(14.7\ in^2)\left(\frac{15.00\ in}{2} + 0.5\ in - 7.13\ in\right)^2 + \frac{1}{12}(7.432\ in^2)(0.5\ in)^3 + (7.432\ in)(0.5\ in)\left(7.13\ in - \frac{0.5\ in}{2}\right)^2$$

$I_x = 1006\ in^4$

c = distance from neutral axis to top of section

$c = 15.00\ in + 0.5\ in - 7.13\ in = 8.37\ in$

Moment capacity of entire section:

$F_y = 36\ ksi$ (assume)

$\phi = 0.9$

$$\phi M_n = \frac{\phi F_y I_x}{c} = \frac{(0.9)(36\ ksi)(1006\ in^4)}{8.37\ in} = 3894\ k\text{-}in = \mathbf{325\ k\text{-}ft}$$

Shear capacity of entire section:

$$\phi V_n = 2(209\ k) = \mathbf{418\ k}$$

Choosing a section: (refer to the spreadsheet in Table D-4)

Frame A:

$M_u^+ = 213\ k\text{-}ft$
$V_u = 34.1\ k$
Choose MC13x31.8 channel with 0.5 *in* thick plate.
$\phi M_n = 221\ k\text{-}ft$
$\phi V_n = 190\ k$

Frame B:

$M_u^+ = 256\ k\text{-}ft$
$V_u = 34.1\ k$
Choose C15x40 channel with 0.5 *in* thick plate.
$\phi M_n = 280\ k\text{-}ft$
$\phi V_n = 304\ k$

Table D-4: Moment and shear capacities of various noncomposite sections (System 1, Option 1).

Shape	*tp* (*in*)	*Phi*Mn* (*k-ft*)	*Phi*Vn* (*k*)
C15x50	0.250	309	418
	0.375	317	418
	0.500	325	418
	0.625	332	418
	0.750	339	418
	0.875	346	418
	1.000	352	418
C15x40	0.250	267	304
	0.375	274	304
	0.500	280	304
	0.625	286	304
	0.750	292	304
	0.875	297	304
	1.000	302	304
C15x33.9	0.250	240	234
	0.375	246	234
	0.500	252	234
	0.625	257	234
	0.750	262	234
	0.875	266	234
	1.000	270	234
MC18x58	0.250	430	490
	0.375	441	490
	0.500	451	490
	0.625	461	490
	0.750	470	490
	0.875	479	490
	1.000	487	490
MC18x51.9	0.250	399	420
	0.375	409	420
	0.500	418	420
	0.625	427	420
	0.750	435	420
	0.875	443	420
	1.000	451	420

Shape	*tp* (*in*)	*Phi*Mn* (*k-ft*)	*Phi*Vn* (*k*)
MC18x45.8	0.250	368	350
	0.375	377	350
	0.500	385	350
	0.625	393	350
	0.750	400	350
	0.875	407	350
	1.000	414	350
MC18x42.7	0.250	352	314
	0.375	361	314
	0.500	369	314
	0.625	376	314
	0.750	383	314
	0.875	390	314
	1.000	396	314
MC13x50	0.250	278	398
	0.375	286	398
	0.500	294	398
	0.625	301	398
	0.750	307	398
	0.875	314	398
	1.000	320	398
MC13x40	0.250	242	284
	0.375	248	284
	0.500	254	284
	0.625	260	284
	0.750	265	284
	0.875	270	284
	1.000	275	284
MC13x35	0.250	223	226
	0.375	228	226
	0.500	234	226
	0.625	239	226
	0.750	243	226
	0.875	247	226
	1.000	252	226

Table D-4 (cont): Moment and shear capacities of various noncomposite sections (System 1, Option 1).

Shape	*tp* (*in*)	*Phi*Mn* (*k-ft*)	*Phi*Vn* (*k*)
MC13x31.8	0.250	211	190
	0.375	216	190
	0.500	221	190
	0.625	225	190
	0.750	229	190
	0.875	233	190
	1.000	237	190
MC12x50	0.250	257	390
	0.375	264	390
	0.500	271	390
	0.625	277	390
	0.750	283	390
	0.875	289	390
	1.000	295	390
MC12x45	0.250	241	332
	0.375	247	332
	0.500	253	332
	0.625	259	332
	0.750	264	332
	0.875	270	332
	1.000	275	332
MC12x40	0.250	224	276
	0.375	229	276
	0.500	235	276
	0.625	240	276
	0.750	245	276
	0.875	249	276
	1.000	254	276
MC12x35	0.250	206	218
	0.375	211	218
	0.500	216	218
	0.625	220	218
	0.750	224	218
	0.875	228	218
	1.000	232	218

Shape	*tp* (*in*)	*Phi*Mn* (*k-ft*)	*Phi*Vn* (*k*)
MC12x31	0.250	193	173
	0.375	197	173
	0.500	201	173
	0.625	205	173
	0.750	209	173
	0.875	212	173
	1.000	215	173

Example 2: Girder configuration option 2

Properties of one channel (C15x50): See Example 1.

Properties of entire section:

$$A = 2(14.7\ in) = 29.4\ in^2$$

$$y_c = \frac{d}{2} = \frac{15.00\ in}{2} = 7.5\ in$$

$$I_x = 2(404\ in^4) = 808\ in^4$$
$$c = 15.00\ in - 7.5\ in = 7.5\ in$$

Moment capacity of entire section:

$F_y = 36\ ksi$ (assume)

$\phi = 0.9$

$$\phi M_n = \frac{(0.9)(36\ ksi)(808\ in^4)}{7.5\ in} = 3491\ k\text{-}in = \mathbf{291\ k\text{-}ft}$$

$\boldsymbol{\phi M_n = 291\ k\text{-}ft}$

Shear capacity of entire section:

$\phi V_n = 2(209\ k) = \mathbf{418\ k}$

Choosing a section: (refer to the spreadsheet in Table D-5)

Frame A:

$M_u^+ = 213\ k\text{-}ft$
$V_u = 34.1\ k$
Choose C15x33.9.
$\phi M_n = 227\ k\text{-}ft$
$\phi V_n = 234\ k$

Frame B:

$M_u^+ = 256\ k\text{-}ft$
$V_u = 34.1\ k$
Choose MC13x50.
$\phi M_n = 261\ k\text{-}ft$
$\phi V_n = 398\ k$

Table D-5: Moment and shear capacities of various noncomposite sections (System 1, Option 2, and System 2).

Shape	*Phi*Mn* (k-ft)	*Phi*Vn* (k)
C15x50	291	418
C15x40	251	304
C15x33.9	227	234
C12x30	146	490
C12x25	130	420
C12x20.7	116	132
C10x30	111	262
C10x25	98.5	204
C10x20	85.2	147
C10x15.3	72.8	93.4
MC18x58	406	490
MC18x51.9	376	420
MC18x45.8	347	350
MC18x42.7	332	314
MC13x50	261	398
MC13x40	227	284
MC13x35	209	226
MC13x31.8	199	190

Shape	*Phi*Mn* (k-ft)	*Phi*Vn* (k)
MC12x50	242	390
MC12x45	227	332
MC12x40	211	276
MC12x35	194	218
MC12x31	183	173
MC10x41.1	171	310
MC10x33.6	150	224
MC10x28.5	137	165
MC10x25	119	148
MC10x22	111	113
MC9x25.4	106	157
MC9x23.9	102	140
MC8x22.8	86.1	133
MC8x21.4	83.2	117
MC8x20	73.6	124
MC8x18.7	70.9	110

Finding the required capacity of the girders after the concrete cures (total load):

In this section, we find the required capacity of the composite girder sections, when the concrete is cured and all the loads will be applied to the structure (dead and live loads). We assume that the beams are simply supported (worst case). Since this is a one-way system, we have a choice as to whether the channel girders run in the short direction of the prototype building (Frame A) or in the long direction (Frame B); see Figure D-2. We check the girders in an interior frame for each of these cases (refer to Table D-2 for the gravity loads we use), for the higher loads from the two load cases shown in Table D-3.

<u>For Frame A</u>: (30 *ft* between frames)

$\omega_{DL} = (65\ psf)(30\ ft) = 1950\ lb/ft = 1.95\ k/ft$ (for floors other than roof)
$\omega_{LL} = (50\ psf)(30\ ft) = 1500\ lb/ft = 1.5\ k/ft$ (for floors other than roof)
$L = 25\ ft$

<u>Load case 1</u>: 1.4 *D*

$$\omega_u = 1.4\ \omega_{DL} = 1.4(1.95\ k/ft) = 2.73\ k/ft$$

$$M_u^+ = \frac{\omega_u L^2}{8} = \frac{(2.73\ k/ft)(25\ ft)^2}{8} = 213\ k\text{-}ft$$

$$V_u = \frac{\omega_u L}{2} = \frac{(2.73\ k/ft)(25\ ft)}{2} = 34.1\ k$$

<u>Load case 2</u>: 1.2 *D* + 1.6 *L*

$$\omega_u = 1.2\ \omega_{DL} + 1.6\ \omega_{LL} = 1.2(1.95\ k/ft) + 1.6(1.5\ k/ft) = 4.74\ k/ft$$

$$M_u^+ = \frac{\omega_u L^2}{8} = \frac{(4.74\ k/ft)(25\ ft)^2}{8} = 370\ k\text{-}ft$$

$$V_u = \frac{\omega_u L}{2} = \frac{(4.74\ k/ft)(25\ ft)}{2} = 59.3\ k$$

<u>Maximum required capacity of composite section</u>:

$\boldsymbol{M_u^+ = 370\ k\text{-}ft}$ (from load case 2)
$\boldsymbol{V_u = 59.3\ k}$ (from load case 2)

<u>For Frame B</u>: (25 *ft* between frames)

$\omega_{DL} = (65\ psf)(25\ ft) = 1625\ lb/ft = 1.625\ k/ft$ (for floors other than roof)
$\omega_{LL} = (50\ psf)(25\ ft) = 1250\ lb/ft = 1.25\ k/ft$ (for floors other than roof)
$L = 30\ ft$

<u>Load case 1</u>: 1.4 *D*

$$\omega_u = 1.4\ \omega_{DL} = 1.4(1.625\ k/ft) = 2.275\ k/ft$$

$$M_u^+ = \frac{\omega_u L^2}{8} = \frac{(2.275\ k/ft)(30\ ft)^2}{8} = 256\ k\text{-}ft$$

$$V_u = \frac{\omega_u L}{2} = \frac{(2.275\ k/ft)(30\ ft)}{2} = 34.1\ k$$

Load case 2: $1.2\ D + 1.6\ L$

$$\omega_u = 1.2\ \omega_{DL} + 1.6\ \omega_{LL} = 1.2(1.625\ k/ft) + 1.6(1.25\ k/ft) = 3.95\ k/ft$$

$$M_u^+ = \frac{\omega_u L^2}{8} = \frac{(3.95\ k/ft)(30\ ft)^2}{8} = 444\ k\text{-}ft$$

$$V_u = \frac{\omega_u L}{2} = \frac{(3.95\ k/ft)(30\ ft)}{2} = 59.3\ k$$

Maximum required capacity of composite section:

$M_u^+ = 444\ k\text{-}ft$ (from load case 2)
$V_u = 59.3\ k$ (from load case 2)

Designing the composite girders (total load):

After finding the required moment and shear capacity of the composite channel girders for each frame option, we need to check the composite strengths of the sections chosen in the noncomposite analysis (see Figures D-5 and D-6 for illustrations of the composite sections with the two girder configuration options). We calculate the moment capacities of the sections based on a plastic stress distribution on the composite section, and we calculate the shear capacities by adding the shear capacities of the component channels (neglecting any contributions from the plate or the slab). We show two examples of these calculations, corresponding to the two girder configuration options. In Example 3, which is applicable to girder configuration option 1, we calculate the moment capacity of a section with C15x50 channels and a 0.5 *in* thick plate; Table D-6 shows the results for other channel sizes and plate thicknesses, which we calculate using a spreadsheet. Then, we check Table D-6 to see if the section we chose in the noncomposite analysis satisfies the moment and shear requirements. In Example 4, which is applicable to girder configuration option 2, we use a similar procedure, calculating the capacity of a section with C15x50 channels and no bottom plate; Table D-7 shows these results for other channel sizes. Then, we check Table D-7 to see if the section we chose in the noncomposite analysis satisfies the moment and shear requirements. Note that we use the length of the girders for Frame A to calculate the effective length of the concrete slab, b_{eff}; for simplicity, we are conservatively using this value for Frame B as well.

General properties:

$F_y = 36\ ksi$
$f'_c = 4\ ksi$
$t_{slab} = 3\ in$

$$b_{eff} = \frac{L}{4} = \frac{25\ ft}{4}\frac{12\ in}{1\ ft} = 75\ in$$

Example 3: Girder configuration option 1

Properties of one channel (C15x50):

$A = 14.7\ in^2$
$d = 15.00\ in$
$t_w = 0.716\ in$ (web thickness)
$b_f = 3.716\ in$ (flange width)

$t_f = 0.650\ in$ (flange thickness)
$I_x = 404\ in^4$
$\phi V_n = 209\ k$

Properties of plate:

$w_p = 2b_f = 2(3.716\ in) = 7.432\ in$
$t_p = 0.5\ in$
$A = (7.432\ in)(0.5\ in) = 3.716\ in^2$

Properties of steel section:

$A = 2(14.7\ in^2) + 3.716\ in^2 = 33.1\ in^2$

$$y_c = \frac{2(14.7\ in^2)\left(\dfrac{15.00\ in}{2} + 0.5\ in\right) + (3.716\ in^2)(0.25\ in)}{33.1\ in^2}$$

$y_c = 7.13\ in$

Case 1: Assuming Plastic Neutral Axis (PNA) is within the concrete slab:

Refer to Figure D-7.

C = resultant compressive force (from compressive concrete)
a = depth of Whitney stress block
T = resultant tensile force (from total steel section)
A_s = total area of steel
$C = 0.85\, f'_c\, a b_{eff}$
$T = A_s F_y$
$C = T$

$$a = \frac{A_s F_y}{0.85 f'_c b_{eff}}$$

If $a < t_{slab}$:

$$d_1 = d + t_p + t_{slab} - \frac{a}{2} - y_c$$

$\phi M_n = \phi T d_1$

For our example:

$$a = \frac{(33.1\ in^2)(36\ ksi)}{0.85(4\ ksi)(75\ in)} = 4.67\ in > 3\ in$$

$a > t_{slab}$, so go to Case 2.

Case 2: Assuming PNA is in the top flanges:

Refer to Figure D-8.

C_c = resultant compressive force from total concrete section
C_s = resultant compressive force from compressive steel
T' = resultant tensile force from tensile steel

A_s = total area of steel
d_f = depth of flange that is in compression
$A_{cs} = 2b_f\, d_f$ = total area of compressive steel

$C_c = 0.85\, f'_c\, b_{eff}\, t_{slab}$
$T' = C_c + C_s$
$T' = A_s F_y - C_s$
$C_c + C_s = A_s F_y - C_s$
$C_s = F_y\, A_{cs} = F_y(2b_f\, d_f)$

$$C_s = \frac{A_s F_y - C_c}{2}$$

$$d_f = \frac{C_s}{2F_y b_f}$$

If $d_f \leq t_f$:

y_{ct} = distance to centroid of tensile steel from bottom of section

$$y_{ct} = \frac{A_s y_c - 2d_f b_f\left(d + t_p - \frac{d_f}{2}\right)}{A_s - 2d_f b_f}$$

y_{cc} = distance to centroid of compressive steel from bottom of section

$$y_{cc} = d + t_p - \frac{d_f}{2}$$

$$d_2' = d + t_p + t_{slab} - \frac{t_{slab}}{2} - y_{ct}$$

$d_2'' = y_{cc} - y_{ct}$
$\phi M_n = \phi(C_c\, d_2' + C_s\, d_2'')$

For our example:

$C_c = 0.85(4\ ksi)(75\ in)(3\ in) = 765\ k$

$$C_s = \frac{(33.1\ in^2)(36\ ksi) - 765\ k}{2} = 213\ k$$

$$d_f = \frac{213\ k}{2(36\ ksi)(3.716\ in)} = 0.797\ in > 0.650\ in$$

$d_f > t_f$, so go to Case 3.

Case 3: Assuming PNA is in the webs:

Refer to Figure D-9.

d_w = depth of web that is in compression
$A_{cs} = 2b_f\, t_f + 2d_w t_w$
$C_c = 0.85\, f'_c\, b_{eff}\, t_{slab}$
$T' = C_c + C_s$
$T' = A_s\, F_y - C_s$
$C_c + C_s = A_s\, F_y - C_s$

$$C_s = \frac{A_s F_y - C_c}{2}$$

$C_s = F_y A_{cs} = F_y (2b_f t_f + 2d_w t_w)$

$$d_w = \frac{C_s - 2F_y b_f t_f}{2F_y t_w}$$

$$y_{ct} = \frac{A_s y_c - 2b_f t_f \left(d + t_p - \frac{t_f}{2}\right) - 2d_w t_w \left(d + t_p - t_f - \frac{d_w}{2}\right)}{A_s - 2b_f t_f - 2d_w t_w}$$

$$y_{cc} = \frac{2b_f t_f \left(d + t_p - \frac{t_f}{2}\right) + 2d_w t_w \left(d + t_p - t_f - \frac{d_w}{2}\right)}{2b_f t_f + 2d_w t_w}$$

$$d_3' = d + t_p + t_{slab} - \frac{t_{slab}}{2} - y_{ct}$$

$d_3'' = y_{cc} - y_{ct}$

$\phi M_n = \phi(C_c d_3' + C_s d_3'')$

For our example:

$C_c = 0.85(4\ ksi)(75\ in)(3\ in) = 765\ k$

$$C_s = \frac{(33.1\ in^2)(36\ ksi) - 765\ k}{2} = 213\ k$$

$$d_w = \frac{213\ k - 2(36\ ksi)(3.716\ in)(0.650\ in)}{2(36\ ksi)(0.716\ in)} = 0.770\ in$$

$y_{ct} = 5.40\ in$

$y_{cc} = 15.04\ in$

$d_3' = 11.60\ in$

$d_3'' = 9.64\ in$

$\phi M_n = (0.9)[(765\ k)(11.60\ in) + (213\ k)(9.64\ in)] = 9840\ k\text{-}in = \mathbf{820\ k\text{-}ft}$

Shear capacity of entire section:

$\phi V_n = 2(209\ k) = \mathbf{418\ k}$

Checking sections chosen based on noncomposite strength:

Refer to the spreadsheet in Table D-6.

Frame A:

$M_u^+ = 370\ k\text{-}ft$
$V_u = 59.3\ k$
MC13x31.8 channel with 0.5 *in* thick plate is OK.
$\phi M_n = 557\ k\text{-}ft$
$\phi V_n = 190\ k$

Frame B:

$M_u^+ = 444\ k\text{-}ft$
$V_u = 59.3\ k$
C15x40 channel with 0.5 *in* thick plate is OK.
$\phi M_n = 706\ k\text{-}ft$
$\phi V_n = 304\ k$

Table D-6: Moment and shear capacities of various composite sections (System 1, Option 1).

Shape	*tp* *(in)*	*Phi*Mn* *(k-ft)*	*Phi*Vn* *(k)*
C15x50	0.250	748	418
	0.375	784	418
	0.500	820	418
	0.625	855	418
	0.750	889	418
	0.875	923	418
	1.000	957	418
C15x40	0.250	634	304
	0.375	670	304
	0.500	706	304
	0.625	742	304
	0.750	778	304
	0.875	814	304
	1.000	850	304
C15x33.9	0.250	559	234
	0.375	594	234
	0.500	629	234
	0.625	664	234
	0.750	700	234
	0.875	735	234
	1.000	771	234
MC18x58	0.250	993	490
	0.375	1038	490
	0.500	1083	490
	0.625	1126	490
	0.750	1169	490
	0.875	1211	490
	1.000	1253	490
MC18x51.9	0.250	919	420
	0.375	966	420
	0.500	1012	420
	0.625	1057	420
	0.750	1102	420
	0.875	1145	420
	1.000	1188	420
MC18x45.8	0.250	835	350
	0.375	883	350
	0.500	931	350
	0.625	979	350
	0.750	1026	350
	0.875	1072	350
	1.000	1118	350
MC18x42.7	0.250	792	314
	0.375	840	314
	0.500	888	314
	0.625	936	314
	0.750	984	314
	0.875	1032	314
	1.000	1078	314
MC13x50	0.250	672	398
	0.375	710	398
	0.500	747	398
	0.625	783	398
	0.750	819	398
	0.875	854	398
	1.000	888	398
MC13x40	0.250	573	284
	0.375	610	284
	0.500	647	284
	0.625	684	284
	0.750	721	284
	0.875	758	284
	1.000	796	284
MC13x35	0.250	520	226
	0.375	556	226
	0.500	592	226
	0.625	629	226
	0.750	666	226
	0.875	703	226
	1.000	740	226

Table D-6 (cont): Moment and shear capacities of various composite sections (System 1, Option 1).

Shape	*tp* *(in)*	*Phi*Mn* *(k-ft)*	*Phi*Vn* *(k)*
MC13x31.8	0.250	485	190
	0.375	521	190
	0.500	557	190
	0.625	593	190
	0.750	630	190
	0.875	667	190
	1.000	703	190
MC12x50	0.250	622	390
	0.375	654	390
	0.500	687	390
	0.625	719	390
	0.750	750	390
	0.875	781	390
	1.000	812	390
MC12x45	0.250	575	332
	0.375	607	332
	0.500	639	332
	0.625	672	332
	0.750	704	332
	0.875	737	332
	1.000	768	332
MC12x40	0.250	530	276
	0.375	562	276
	0.500	594	276
	0.625	626	276
	0.750	658	276
	0.875	690	276
	1.000	722	276
MC12x35	0.250	481	218
	0.375	512	218
	0.500	544	218
	0.625	575	218
	0.750	606	218
	0.875	638	218
	1.000	670	218

Shape	*tp* *(in)*	*Phi*Mn* *(kip-ft)*	*Phi*Vn* *(k)*
MC12x31	0.250	441	173
	0.375	472	173
	0.500	503	173
	0.625	534	173
	0.750	565	173
	0.875	596	173
	1.000	628	173

Example 4: Girder configuration option 2

Properties of one channel (C15x50): See Example 3.

Properties of steel section:

$$A = 2(14.7\ in^2) = 29.4\ in^2$$

$$y_c = \frac{d}{2} = \frac{15.00\ in}{2} = 7.5\ in$$

Case 1: Assuming Plastic Neutral Axis (PNA) is within the concrete slab:

Refer to Figure D-10.

$$a = \frac{A_s F_y}{0.85 f'_c b_{eff}} \qquad \text{(as in Example 3)}$$

If $a < t_{slab}$:

$$d_1 = d + t_{slab} - \frac{a}{2} - y_c$$

$$\phi M_n = \phi T d_1$$

For our example:

$$a = \frac{(29.4\ in^2)(36\ ksi)}{0.85(4\ ksi)(75\ in)} = 4.15\ in > 3\ in$$

$a > t_{slab}$, so go to Case 2.

Case 2: Assuming PNA is in the top flanges:

Refer to Figure D-11.

$$d_f = \frac{C_s}{2F_y b_f} \qquad \text{(as in Example 3)}$$

If $d_f < t_f$:

$$y_{ct} = \frac{A_s y_c - 2d_f b_f \left(d - \frac{d_f}{2}\right)}{A_s - 2d_f b_f}$$

$$y_{cc} = d + t_p - \frac{d_f}{2}$$

$$d_2' = d + t_{slab} - \frac{t_{slab}}{2} - y_{ct}$$

$$d_2'' = y_{cc} - y_{ct}$$

$$\phi M_n = \phi(C_c\, d_2' + C_s\, d_2'')$$

For our example:

$C_c = 765\ k$
$C_s = 147\ k$
$d_f = 0.548\ in < 0.650\ in$
(If $d_f > t_f$ then we would go to case 3.)
$y_{ct} = 6.337\ in$
$y_{cc} = 14.91\ in$
$d_3{}' = 10.163\ in$
$d_3{}'' = 8.573\ in$
$\phi M_n = (0.9)[(765\ k)(10.163\ in) + (146.7\ k)(8.573\ in)] = 8129\ k\text{-}in = \mathbf{677\ k\text{-}ft}$

Case 3: Assuming PNA is in the webs:

Refer to Figure D-12.

$$d_w = \frac{C_s - 2F_y b_f t_f}{2F_y t_w} \qquad \text{(as in Example 3)}$$

$$y_{ct} = \frac{A_s y_c - 2b_f t_f\left(d - \frac{t_f}{2}\right) - 2d_w t_w\left(d - t_f - \frac{d_w}{2}\right)}{A_s - 2b_f t_f - 2d_w t_w}$$

$$y_{cc} = \frac{2b_f t_f\left(d - \frac{t_f}{2}\right) + 2d_w t_w\left(d - t_f - \frac{d_w}{2}\right)}{2b_f t_f + 2d_w t_w}$$

$$d_3{}' = d + t_{slab} - \frac{t_{slab}}{2} - y_{ct}$$
$d_3{}'' = y_{cc} - y_{ct}$
$\phi M_n = \phi(C_c\, d_3{}' + C_s\, d_3{}'')$

Shear capacity of entire section:

$\phi V_n = 2(209\ k) = \mathbf{418\ k}$

Checking sections chosen based on noncomposite strength:

Refer to the spreadsheet in Table D-7.

Frame A:

$M_u{}^+ = 370\ k\text{-}ft$
$V_u = 59.3\ k$
C15x33.9 is OK.
$\phi M_n = 489\ k\text{-}ft$
$\phi V_n = 234\ k$

Frame B:

$M_u{}^+ = 444\ k\text{-}ft$
$V_u = 59.3\ k$
MC13x50 is OK.
$\phi M_n = 597\ k\text{-}ft$
$\phi V_n = 398\ k$

Table D-7: Moment and shear capacities of various composite sections (System 1, Option 2, and System 2).

Shape	*Phi*Mn* (*k-ft*)	*Phi*Vn* (*k*)
C15x50	675	418
C15x40	563	304
C15x33.9	489	234
C12x30	369	490
C12x25	316	420
C12x20.7	268	132
C10x30	322	262
C10x25	276	204
C10x20	228	147
C10x15.3	179	93.4
MC18x58	901	490
MC18x51.9	822	420
MC18x45.8	739	350
MC18x42.7	697	314
MC13x50	597	398
MC13x40	500	284
MC13x35	448	226
MC13x31.8	413	190

Shape	*Phi*Mn* (*k-ft*)	*Phi*Vn* (*k*)
MC12x50	557	390
MC12x45	512	332
MC12x40	468	276
MC12x35	420	218
MC12x31	380	173
MC10x41.1	412	310
MC10x33.6	352	224
MC10x28.5	308	165
MC10x25	276	148
MC10x22	247	113
MC9x25.4	260	157
MC9x23.9	247	140
MC8x22.8	219	133
MC8x21.4	207	117
MC8x20	196	124
MC8x18.7	185	110

JOISTS

We also need to determine the size and number of joists that will be included in the preassembled panel. The procedure we use is to choose a number of joists per panel, to calculate the live and total loads that would act on the joists corresponding to the joist spacing, and to refer to Vulcraft's "Steel Joists and Joist Girders Design Manual (1991)" to choose a section. If the loads are too high or the chosen section is too deep, we increase the number of joists per panel and repeat the process. We also determine whether we need to size up the channels previously chosen to accommodate the depths of the joists. Example 5 illustrates the calculation of the required number and sizes of joists, for Frame A. The calculation of the required number of joists for Frame B is similar; Table 5-8 summarizes the results for the two frame options and two girder configurations. Once we determine the number and sizes of the joists and redesigned channels, we calculate the weights of the panels, which are illustrated in Example 6 and summarized in Table 5-9 for the various panel options.

Example 5: Calculating number and sizes of joists, Frame A

Loads:

Dead load: 65 *psf* Total load: 115 *psf*
Live load: 50 *psf* $L = 30\ ft$

Try 9 joists per panel:

LIVE = (50 *psf*)(25 *ft*)/9 = 139 *lb/ft*
TOTAL = (115 *psf*)(25 *ft*)/9 = 320 *lb/ft*
Choose 16K9.
$d = 16\ in$ (Channels must be sized up to accommodate these joists.)
weight = 10 *lb/ft*
LIVE CAPACITY = 178 *lb/ft*
TOTAL CAPACITY = 355 *lb/ft*

Redesigning channels to accommodate joist depth:

Girder configuration option 1:

Choose MC18x42.7. (Refer to Tables D-4 and D-6.)

Girder configuration option 2:

Choose MC18x42.7. (Refer to Tables D-5 and D-7.)

Table D-8: Joist results.

	Configuration Option 1		**Configuration Option 2**	
	Frame A	**Frame B**	**Frame A**	**Frame B**
# of joists	9	7	9	7
Joist designation	16K9	16K9	16K9	16K9
Weight of joists	10 *lb/ft*	10 *lb/ft*	10 *lb/ft*	10 *lb/ft*
Redesigned channels	MC18x42.7	MC18x42.7	MC18x42.7	MC18x42.7

Example 6: Calculating panel weight, Girder configuration option 1, Frame A

Weight of each girder:

$L = 25\ ft$
$weight = (42.7\ lb/ft + 6.81\ \text{lb/ft})(25\ ft) = 1238\ lb$

Weight of each joist:

$L = 30\ ft$
$weight = (10\ lb/ft)(30\ ft) = 300\ lb$

Total weight of panel (excluding metal decking):

$weight = 2(1238\ lb) + 9(300\ lb) = 5176\ lb = 5.2\ tons$

Table D-9: Panel weights.

	Configuration Option 1		Configuration Option 2	
	Frame A	**Frame B**	**Frame A**	**Frame B**
Weight/panel	5.2 *tons*	4.7 *tons*	4.8 *tons*	4.3 *tons*

GIRDER TO COLUMN CONNECTION

After the columns and girders are chosen, it is necessary to design the girder to column connection (panel to column connection). For this system, we must design the seat angles on which the girders (panels) sit, also determining whether the seat angles must be stiffened (a top angle must also be placed for lateral support of the compression flange). Figures D-13 and D-14 illustrate this connection for the two girder configuration options.

Finding the required capacity of the connections:

Since we assume that the girders are simply supported, this is a simple (type 2) connection which we design to carry only shear. The shear forces that are transferred from the girders to the columns (which have the same values regardless of whether Frame A or Frame B is chosen) were calculated previously when we designed the girders for the total loads:

$$P_u = 59.3\ k$$

Designing the seated beam connections:

After finding the maximum required shear capacity of the connection, we must design the seating angle. This depends on the size and configuration of the girder, in addition to the shear force that is transferred to the columns. In Example 6, we calculate the size of the seating angle for girder configuration option 1 in Frame A. The calculations for the other options are similar to those shown in Example 7; Table D-10 summarizes the results for the four options.

Example 7: Calculating size of seating angle, Girder configuration option 1, Frame A

Properties of one channel (MC18x42.7):

$d = 18.00\ in$
$t_w = 0.450\ in$
$b_f = 3.950\ in$
$t_f = 0.625\ in$

$k = 1.375\ in$ (distance from outer face of flange to web toe of fillet)

Properties of plate:

$t_p = 0.50\ in$

Properties of section:

$d = 18.00\ in + 0.50\ in = 18.50\ in$
$t_w = 2(0.450\ in) = 0.900\ in$
$t_f = 0.625\ in + 0.50\ in = 1.125\ in$
$k = 1.375\ in$
½ *in* nominal beam setback
$F_y = 36\ ksi$ (for both beam and seat angle)

Calculating required bearing length based on local web yielding:

$$N = \frac{P_u}{\phi F_{yw} t_w} - 2.5k$$

$\phi = 1.0$

$$N = \frac{59.3k}{(1.0)(36ksi)(0.90in)} - 2.5(1.375in) = negative$$

Use a minimum bearing length of 4 *in* (angle leg).

Checking web crippling:

Assuming 0.5 *in* angle thickness, use $N = 4.0\ in - 0.5\ in = 3.5\ in$.
$\phi = 0.75$

$$\phi P_n = \phi\, 68 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi P_n = (0.75)(68)(0.9\ in)^2 \left[1 + 3\left(\frac{3.5\ in}{18.5\ in}\right)\left(\frac{0.9\ in}{1.125\ in}\right)^{1.5}\right] \cdot \sqrt{\frac{(36\ ksi)(1.125\ in)}{0.9\ in}}$$

$\phi P_n = 315\ k > P_u = 59.3\ k$, OK.

Finding angle thickness:

$N = k$ is used for determining the angle thickness.
$N = 1.375\ in$
erection clearance = 0.75 *in*

$$e_f = erection\ clearance + \frac{N}{2}$$

$$e_f = 0.75\ in + \frac{1.375\ in}{2} = 1.4375\ in$$

L = angle length (allowing for minimum required weld end return)

$L = 2b_f + 1.5\ in = 2\ (3.95\ in) + 1.5\ in = 9.40\ in$

Try $t = 0.5\ in$:

$e = e_f - t - 0.375\ in$
$e = 1.4375\ in - 0.5\ in - 0.375\ in = 0.5625\ in$
$\phi = 0.90$

$$t^2 = \frac{4P_u e}{\phi F_y L}$$

$$t^2 = \frac{4(59.3\ k)(0.5625\ in)}{(0.90)(36\ ksi)(9.4\ in)} = 0.438\ in^2$$

$t = 0.662\ in \rightarrow Use\ t = 0.75\ in$

Use seat angle, 0.75 *in* thick and 9.5 *in* long with 4 *in* leg.

Determining welded connection to column:

E70 electrodes, shielded metal arc welding process
column thickness = 0.375 *in*
angle thickness = 0.75 *in* (thicker)
$a_{min} = 0.25\ in$ (from AISC specification, based on angle thickness)
$a_{max} = 0.75\ in - 0.0625\ in = 0.6875\ in$

Try $L = 4\ in$ supported leg:

$$R_u = \frac{P_u}{2L^2}\sqrt{L^2 + 20.25 e_1^2}$$

$e_l = e_f$

$$R_u = \frac{59.3\ k}{2(4\ in)^2}\sqrt{(4\ in)^2 + 20.25(1.4375\ in)^2} = 14.1\ k/in$$

$\phi R_{nw} = \phi(0.707a)(0.60F_{EXX})$
$\phi = 0.75$
$\phi R_{nw} = (0.75)(0.707)a(0.60)(70\ ksi) = (22.3\ ksi)a$
$R_u = \phi R_{nw}$

$$a = \frac{14.1\ k/in}{22.3\ ksi} = 0.633\ in \rightarrow Use\ a = 0.6875\ in$$

Try $L = 6\ in$ supported leg:

$$R_u = \frac{59.3\ k}{2(6\ in)^2}\sqrt{(6\ in)^2 + 20.25(1.4375\ in)^2} = 7.27\ k/in$$

$\phi R_{nw} = (22.3\ ksi)a$

$$a = \frac{7.27\ k/in}{22.3\ ksi} = 0.326\ in \rightarrow Use\ a = 0.375\ in$$

Use 6 *in* supported leg, with ⅜ *in* weld.

Note that an unstiffened angle is acceptable, so a stiffened angle is unnecessary.

Table D-10: Girder to column connection results.

	Configuration Option 1		Configuration Option 2	
	Frame A	Frame B	Frame A	Frame B
Required Seating Angle	L6x4x¾ 0'-9.4" long	L6x4x¾ 0'-9.4" long	L6x4x¾ 0'-9.4" long	L6x4x¾ 0'-9.4" long
Required Weld size, a	⅜ *in*	⅜ *in*	⅜ *in*	⅜ *in*

STRUCTURAL ANALYSIS OF SYSTEM 2

COLUMNS

Finding the required capacity of the columns:

As with System 1, we first determine the required axial capacity for an interior column on the first floor (subjected to gravity loads only), using the same procedure and the same loads and load cases, which are listed in Tables D-2 and D-3, respectively. To find the axial force in a first-story interior column (worst case), for each of the load cases we sum the forces acting on the tributary areas of the columns for all of the floors above the first story (with the appropriate load factors), and we choose the highest load. The resulting maximum required axial capacity of the columns is the same value as for System 1:

$P_u = 529\ k$

Designing the columns:

After finding the maximum required axial capacity of a first-story interior column, we refer to the LRFD manual to choose a W-shape column.

Assumptions:

$P_u = 529\ k$
$K = 1.2$ (assuming rotation fixed, translation free)
$L = 12\ ft$ (story height)
$KL = (1.2)(12\ ft) = 14.4\ ft$ (effective length)

From LRFD Manual:

Choose W12x79.
$\phi P_n = 599\ k$ (assuming $F_y = 36\ ksi$)

GIRDERS--Beam stubs and channel sections

Finding the required capacity of the girders before the concrete cures (dead loads only):

Because the girders do not act compositely until the concrete cures, it is necessary to find the required capacity of the steel sections subjected to dead loads before the concrete cures. Since this system uses columns with rigidly attached beam stubs, we assume that the girders are fixed, and that the connections between beam stubs and channel sections are made at the moment inflection points. Since this is a one-way system, we have a choice as to whether the channel girders run in the short direction of the prototype building (Frame A) or in the long direction (Frame B); see Figure D-2. As with System 1, we check the girders in an interior frame for each of these cases (refer to Table D-2 for the gravity loads we use); this time we find the maximum positive and negative moments to check both the beam stub and channel section.

For Frame A: (30 ft between frames)

$\omega_{DL} = (65\ psf)(30\ ft) = 1950\ lb/ft = 1.95\ k/ft$ (for floors other than roof)
$\omega_u = 1.4\ \omega_{DL} = 1.4(1.95\ k/ft) = 2.73\ k/ft$
$L = 25\ ft$

$$M_u^+ = \frac{\omega L^2}{24} = \frac{(2.73\ k/ft)(25\ ft)^2}{24} = \mathbf{71.1\ k\text{-}ft}$$

$$M_u^- = \frac{\omega L^2}{12} = \frac{(2.73\ k/ft)(25\ ft)^2}{12} = \mathbf{142\ k\text{-}ft}$$

$$V_u = \frac{\omega L}{2} = \frac{(2.73\ k/ft)(25\ ft)}{2} = \mathbf{34.1\ k}$$

For Frame B: (25 ft between frames)

$\omega_{DL} = (65\ psf)(25\ ft) = 1625\ lb/ft = 1.625\ k/ft$ (for floors other than roof)
$\omega_u = 1.4\ \omega_{DL} = 1.4(1.625\ k/ft) = 2.275\ k/ft$
$L = 30\ ft$

$$M_u^+ = \frac{\omega L^2}{24} = \frac{(2.275\ k/ft)(30\ ft)^2}{24} = \mathbf{85.3\ k\text{-}ft}$$

$$M_u^- = \frac{\omega L^2}{12} = \frac{(2.275\ k/ft)(30\ ft)^2}{12} = \mathbf{171\ k\text{-}ft}$$

$$V_u = \frac{\omega L}{2} = \frac{(2.275\ k/ft)(30\ ft)}{2} = \mathbf{34.1\ k}$$

Designing the beam stubs (dead loads only):

After finding the required moment and shear capacity of the beam stubs before the concrete cures for each frame option, we need to choose an appropriate W-shape section. We choose a section for the beam stub based on satisfying the required section modulus, and we check the shear capacity.

Frame A:

$M_u^- = 142\ k\text{-}ft$
$V_u = 34.1\ k$
$\phi M_n = \phi F_y S_x$
$\phi = 0.90$

$$S_{xreq} = \frac{M_u^-}{\phi F_y} = \frac{142\ k\text{-}ft}{(0.9)(36\ ksi)} \cdot \frac{12\ in}{1\ ft} = 52.7\ in^3$$

Choose W14x38.
$S_x = 54.6\ in^3$
$\phi M_n = (0.9)(36\ ksi)(54.6\ in^3) = 1770\ k\text{-}in = 148\ k\text{-}ft$
$\phi V_n = 85.0\ k$ (OK)

Frame B:
$M_u^- = 171\ k\text{-}ft$
$V_u = 34.1\ k$

$$S_{xreq} = \frac{171\ k\text{-}ft}{(0.9)(36\ ksi)} \cdot \frac{12\ in}{1\ ft} = 63.2\ in^3$$

Choose W14x48.

$S_x = 70.3\ in^3$

$\phi M_n = (0.9)(36\ ksi)(70.3\ in^3) = 2280\ k\text{-}in = 190\ k\text{-}ft$

$\phi V_n = 91.1\ k$ (OK)

Designing the noncomposite channel sections (dead loads only):

After finding the required moment and shear capacity of the noncomposite channel girders for each frame option, we need to choose an appropriate section, which includes two back-to-back channels, as shown in Figure D-4. To choose a channel section, we use the results of the analysis of girder configuration option 2 of System 1, where we calculated the capacities of noncomposite sections with different channel shapes (refer to Table D-5), and we choose one with sufficient capacity. We calculated the capacities of the channel sections based on first yielding of the exterior fibers of the section, assuming that the spanning elements provide enough lateral stability to avoid lateral torsional buckling. We calculated the shear capacities by adding the shear capacities of the component channels (neglecting any contributions from the plate). The sections chosen to satisfy this noncomposite strength requirement must, of course, also have enough strength with the full load application (dead and live loads) when the channel section acts compositely (when the concrete cures); we will check this in the next section.

Choosing a section: (refer to Table D-5)

Frame A:

$M_u^+ = 71.1\ k\text{-}ft$

$V_u = 34.1\ k$

Choose C10x15.3.

$\phi M_n = 72.8\ k\text{-}ft$

$\phi V_n = 93.4\ k$

Frame B:

$M_u^+ = 85.3\ k\text{-}ft$

$V_u = 34.1\ k$

Choose MC8x22.8.

$\phi M_n = 86.1\ k\text{-}ft$

$\phi V_n = 133\ k$

(Note: The channels may have to be redesigned if they are not deep enough to accommodate the splice connection to the beam stub.)

Finding the required capacity of the girders after the concrete cures (total load):

In this section, we find the required capacity of the beam stub and composite channel section when the concrete is cured and all the loads will be applied to the structure (dead and live loads). Since this system uses columns with rigidly attached beam stubs, we assume that the girders are fixed, and that the connections between beam stubs and channel sections are made at the moment inflection points. Since this is a one-way system, we have a choice as to whether the channel girders run in the short direction of the prototype building (Frame A) or in the long direction (Frame B); see Figure D-2. As with System 1, we check the girders in an interior frame for each of these cases (refer to Table D-2 for the gravity loads we use), for the higher loads from the two load cases in shown in Table D-3; this time we find the maximum positive and negative moments to check both the beam stub and channel section.

For Frame A: (30 *ft* between frames)

$\omega_{DL} = (65\ psf)(30\ ft) = 1950\ lb/ft = 1.95\ k/ft$ (for floors other than roof)

$\omega_{LL} = (50\ psf)(30\ ft) = 1500\ lb/ft = 1.5\ k/ft$ (for floors other than roof)

$L = 25\ ft$

Load case 1: 1.4 D

$$\omega_u = 1.4\ \omega_{DL} = 1.4(1.95\ k/ft) = 2.73\ k/ft$$

$$M_u^+ = \frac{\omega L^2}{24} = \frac{(2.73\ k/ft)(25\ ft)^2}{24} = 71.1\ k\text{-}ft$$

$$M_u^- = \frac{\omega L^2}{12} = \frac{(2.73\ k/ft)(25\ ft)^2}{12} = 142\ k\text{-}ft$$

$$V_u = \frac{\omega L}{2} = \frac{(2.73\ k/ft)(25\ ft)}{2} = 34.1\ k$$

Load case 2: 1.2 D + 1.6 L

$$\omega_u = 1.2\ \omega_{DL} + 1.6\ \omega_{LL} = 1.2(1.95\ k/ft) + 1.6(1.5\ k/ft) = 4.74\ k/ft$$

$$M_u^+ = \frac{\omega L^2}{24} = \frac{(4.74\ k/ft)(25\ ft)^2}{24} = 123\ k\text{-}ft$$

$$M_u^- = \frac{\omega L^2}{12} = \frac{(4.74\ k/ft)(25\ ft)^2}{12} = 247\ k\text{-}ft$$

$$V_u = \frac{\omega L}{2} = \frac{(4.74\ k/ft)(25\ ft)}{2} = 59.3\ k$$

Maximum required capacity of (noncomposite) beam stub:

$M_u^- = 246.9\ k\text{-}ft$ (from load case 2)
$V_u = 59.3\ k$ (from load case 2)

Maximum required capacity of composite section:
$M_u^+ = 123.4\ k\text{-}ft$ (from load case 2)
$V_u = 59.3\ k$ (from load case 2)

For Frame B: (25 ft between frames)

$\omega_{DL} = (65\ psf)(25\ ft) = 1625\ lb/ft = 1.625\ k/ft$ (for floors other than roof)
$\omega_{LL} = (50\ psf)(25\ ft) = 1250\ lb/ft = 1.25\ k/ft$ (for floors other than roof)
$L = 30\ ft$

Load case 1: 1.4 D

$$\omega_u = 1.4\ \omega_{DL} = 1.4(1.625\ k/ft) = 2.275\ k/ft$$

$$M_u^+ = \frac{\omega L^2}{24} = \frac{(2.275\ k/ft)(30\ ft)^2}{24} = 85.3\ k\text{-}ft$$

$$M_u^- = \frac{\omega L^2}{12} = \frac{(2.275\ k/ft)(30\ ft)^2}{12} = 171\ k\text{-}ft$$

$$V_u = \frac{\omega L}{2} = \frac{(2.275\ k/ft)(30\ ft)}{2} = 34.1\ k$$

Load case 2: 1.2 D + 1.6 L

$$\omega_u = 1.2\ \omega_{DL} + 1.6\ \omega_{LL} = 1.2(1.625\ k/ft) + 1.6(1.25\ k/ft) = 3.95\ k/ft$$

$$M_u^+ = \frac{\omega L^2}{24} = \frac{(3.95\ k/ft)(30\ ft)^2}{24} = 148\ k\text{-}ft$$

$$M_u^- = \frac{\omega L^2}{12} = \frac{(3.95\ k/ft)(30\ ft)^2}{12} = 296\ k\text{-}ft$$

$$V_u = \frac{\omega L}{2} = \frac{(3.95\ k/ft)(30\ ft)}{2} = 59.3\ k$$

Maximum required capacity of (noncomposite) beam stub:

$\boldsymbol{M_u^- = 296\ k\text{-}ft}$	(from load case 2)
$\boldsymbol{V_u = 59.3\ k}$	(from load case 2)

Maximum required capacity of composite section:

$\boldsymbol{M_u^+ = 148\ k\text{-}ft}$	(from load case 2)
$\boldsymbol{V_u = 59.3\ k}$	(from load case 2)

Designing the beam stubs (total loads):

After finding the required moment and shear capacity of the beam stubs after the concrete cures for each frame option, we need to choose an appropriate W-shape section. We choose a section for the beam stub based on satisfying the required section modulus, and we check the shear capacity. Since the beam stub is in a negative moment region, the concrete strength cannot be counted on, and the beam stub acts noncompositely.

Frame A:

$M_u^- = 247\ k\text{-}ft$
$V_u = 59.3\ k$

$$S_{xreq} = \frac{247\ k\text{-}ft}{(0.9)(36\ ksi)} \cdot \frac{12\ in}{1\ ft} = 91.4\ in^3$$

Choose W14x68.
$S_x = 103\ in^3$
$\phi M_n = (0.9)(36\ ksi)(103\ in^3) = 3337\ k\text{-}in = 278\ k\text{-}ft$
$\phi V_n = 113\ k$ (OK)

Frame B:

$M_u^- = 296\ k\text{-}ft$
$V_u = 59.3\ k$

$$S_{xreq} = \frac{296\ k\text{-}ft}{(0.9)(36\ ksi)} \cdot \frac{12\ in}{1\ ft} = 110\ in^3$$

Choose W14x74.
$S_x = 112\ in^3$
$\phi M_n = (0.9)(36\ ksi)(112\ in^3) = 3629\ k\text{-}in = 302\ k\text{-}ft$
$\phi V_n = 124\ k$ (OK)

Designing the composite channel sections (total loads):

After finding the required moment and shear capacity of the composite channel girders for each frame option, we need to check the composite strengths of the sections chosen in the noncomposite analysis (see Figure D-6 for an illustration of the composite section). To do this, we use the results of the analysis of girder configuration option 2 of System 1, where we calculated the capacities of composite sections with different channel shapes (refer to Table D-7), and we check the capacities of the chosen sections. We calculated the moment capacities of the sections based on a plastic stress distribution on the composite section, and we calculated the shear capacities by adding the shear capacities of the component channels (neglecting any contributions from the plate or the slab).

Checking sections chosen based on noncomposite strength:

Refer Table D-7.

Frame A:

$M_u^+ = 123$ *k-ft*
$V_u = 59.3$ *k*
C10x15.3 is OK.
$\phi M_n = 179$ *k-ft*
$\phi V_n = 93.4$ *k-ft*

Frame B:

$M_u^+ = 148$ *k-ft*
$V_u = 59.3$ *k*
MC8x22.8 is OK.
$\phi M_n = 219$ *k-ft*
$\phi V_n = 133$ *k-ft*

JOISTS

We also need to determine the size and number of joists that will be included in the preassembled panel. The procedure we use is the same as that we describe for System 1, although for this system, some of the joists are stick-built and not included in the preassembled panel, which is determined by the length of the beam stubs. We also determine whether the joists must be sized up to accommodate the depth of the joists. Table 5-11 summarizes the results for the two frame options. Once we determine the number and sizes of the joists and redesigned channels, we calculate the weights of the panels, which are summarized in Table 5-12 for the various panel options.

Table D-11: Joist results.

	Frame A	**Frame B**
Total # of joists	9	7
# of joists per panel	5	3
# of stick-built joists per bay	4	4
Joist designation	16K9	16K9
Weight of joists	10 *lb/ft*	10 *lb/ft*
Redesigned channels	MC18x42.7	MC18x42.7

Table D-12: Panel weights.

	Frame A	**Frame B**
Weight/panel	2.7 *tons*	2.2 *tons*

GIRDER SPLICE CONNECTION (Beam stub to channel section)

After the members are chosen, it is necessary to design the splice between the beam stub and the channel section. For this system, we design the number of bolts that are necessary to transfer the loads between the two sections. Figure D-15 illustrates this connection.

Finding the required capacity of the connections:

Since the splice is theoretically located at the moment inflection point, we design the connection to have adequate shear capacity; we assume that there are top and bottom plates that will transfer any moment that may develop, although we do not design this part of the connection. The shear forces that are transferred between the beam stub and the channel section (which have the same values regardless of whether Frame A or Frame B is chosen) were calculated previously when we designed the girders for the total loads:

$$V_u = 59.3\ k$$

Designing the splice connections:

After finding the maximum required shear capacity of the connection, we determine the number of bolts that are necessary to transfer that shear. In Example 8, we show this calculation for a girder splice in either Frame A or Frame B. It should be noted that in order to use top and bottom plates, the beam stubs need to be sized up to the depth of the redesigned channel girders. Table D-13 shows the required number of bolts and redesigned beam stubs for the two frame options.

Example 8: Calculating the number of bolts necessary to transfer shear

Properties of each bolt:

Assume ¾ *in* diameter bolts.
$F_u^b = 120\ ksi$
$m = 2$ (number of shear planes)

Shear capacity of each bolt:

$$\phi R_n = \phi(0.60 F_u^b) m A_b$$
$$\phi = 0.65$$
$$\phi R_n = (0.65)(0.60)(120\ ksi)(2)[(\pi/4)(0.75\ in)^2] = 41.4\ k$$

Required number of bolts:

$$number\ of\ bolts = \frac{V_u}{\phi R_n} = \frac{59.3\ k}{41.4\ k} = 2$$

Use 2 bolts on each side of shear splice.

Table D-13: Girder splice connection results.

	Frame A	**Frame B**
Total # of bolts	2	2
Redesigned beam stubs	W18x55	W18x60

Plan View

N

Elevation View
(Looking North)

Elevation View
(Looking East)

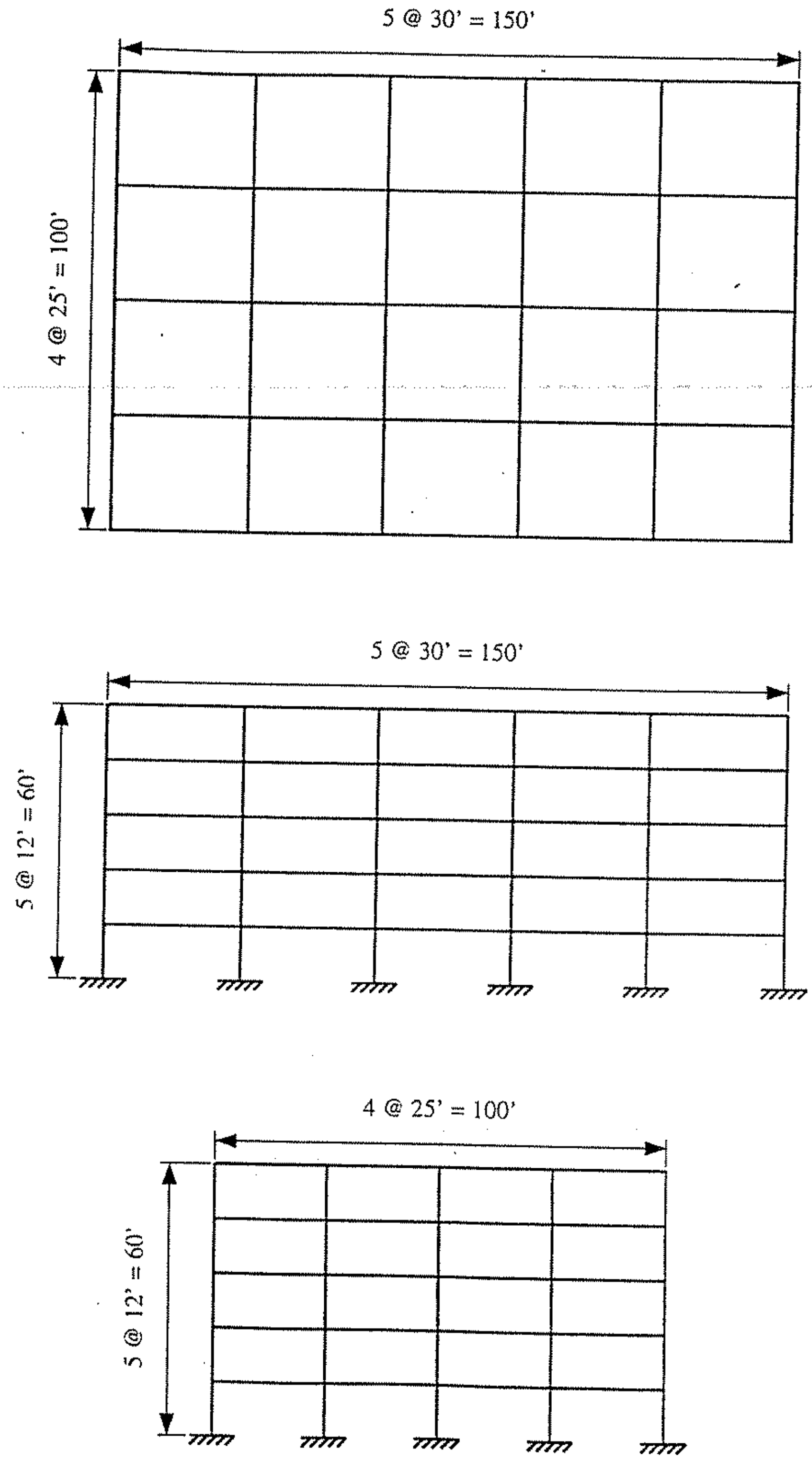
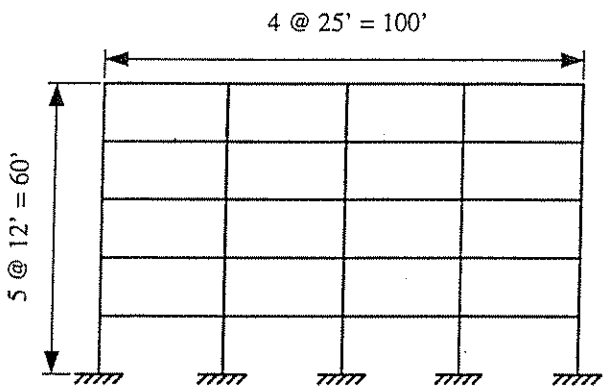
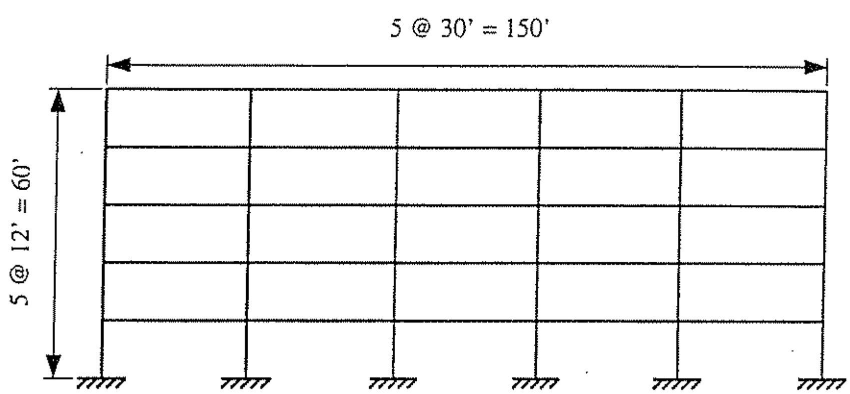


Figure D-1: Prototype Building.



Frame A



Frame B

Figure D-2: Frame Options.

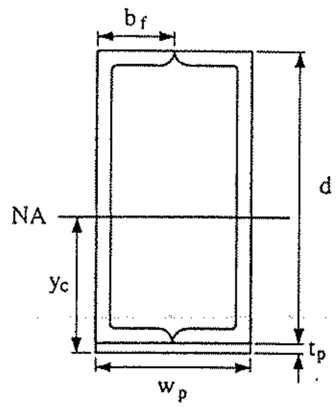


Figure D-3: Noncomposite girder section (System 1, Option 1).

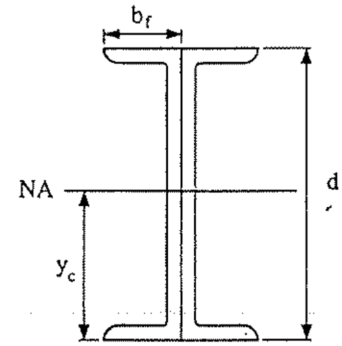


Figure D-4: Noncomposite girder section (System 1, Option 2, and System 2).

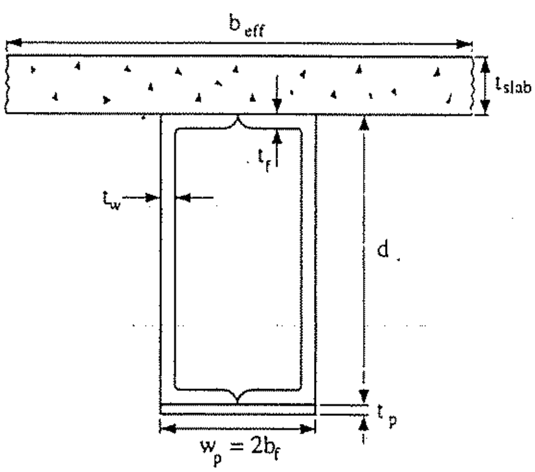


Figure D-5: Composite girder section (System 1, Option 1).

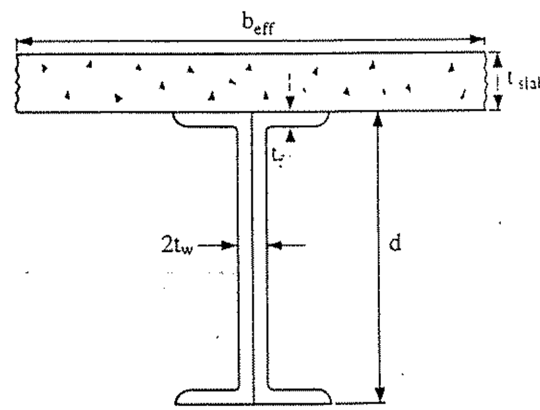


Figure D-6: Composite girder section (System 1, Option 2, and System 2).

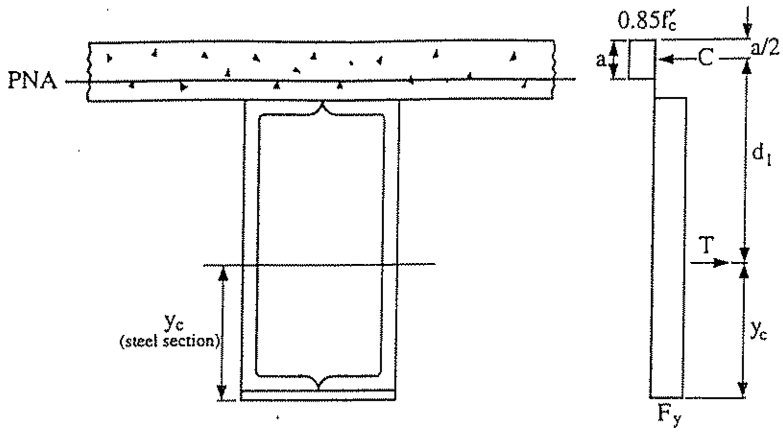


Figure D-7: Analysis of composite girder section--Case 1 (System 1, Option 1).

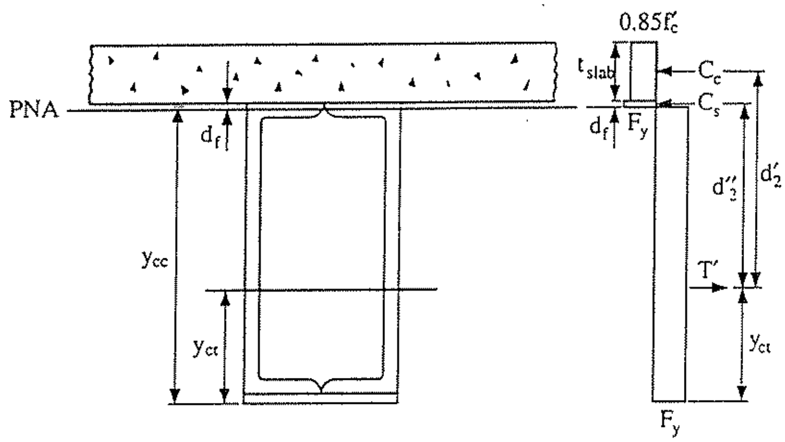


Figure D-8: Analysis of composite girder section--Case 2 (System 1, Option 1).

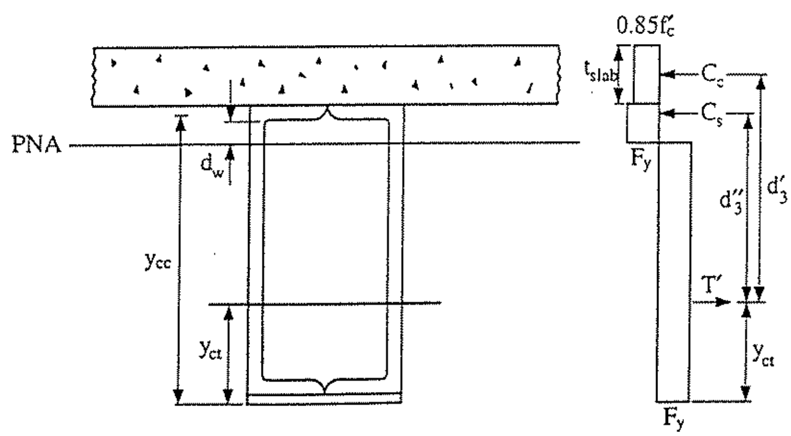


Figure D-9: Analysis of composite girder section--Case 3 (System 1, Option 1).

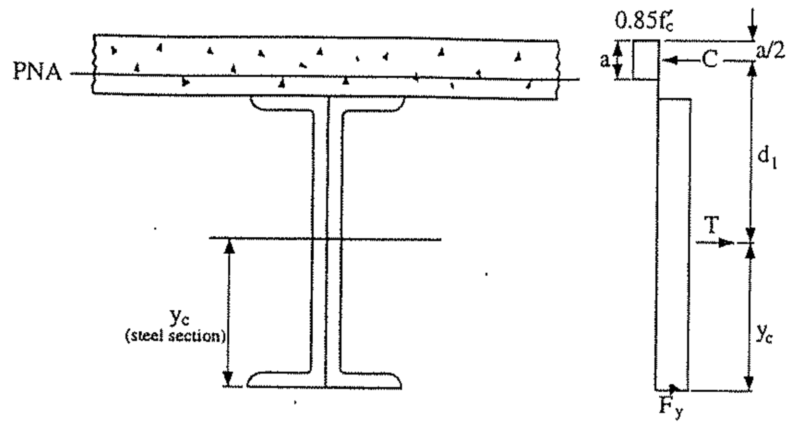


Figure D-10: Analysis of composite girder section--Case 1 (System 1, Option 2, and System 2).

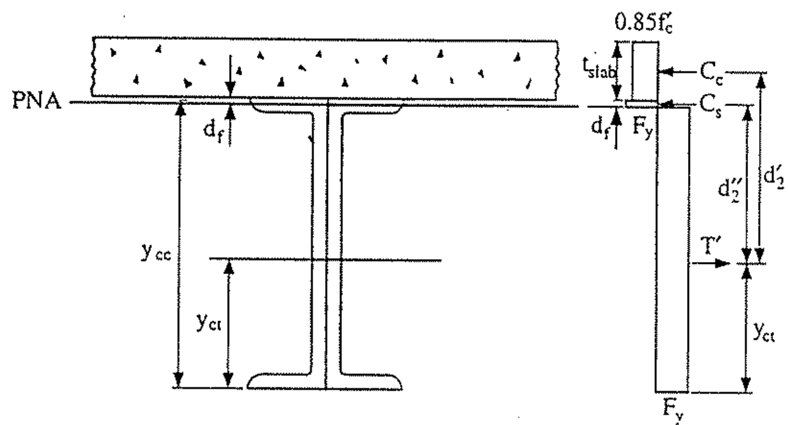


Figure D-11: Analysis of composite girder section--Case 2 (System 1, Option 2, and System 2).

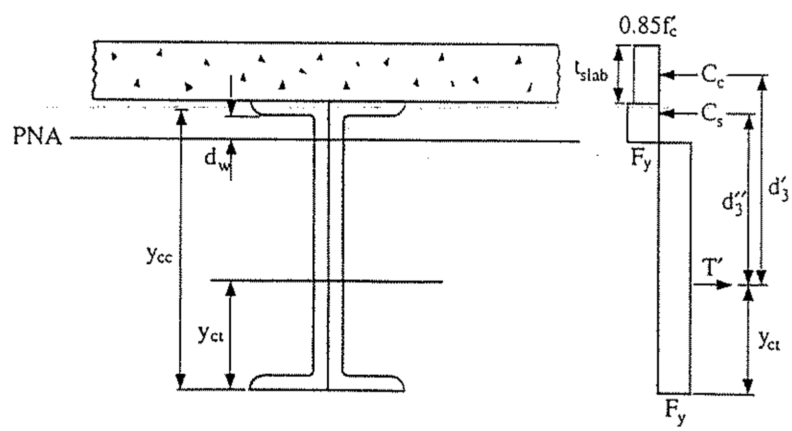


Figure D-12: Analysis of composite girder section--Case 3 (System 1, Option 2, and System 2).

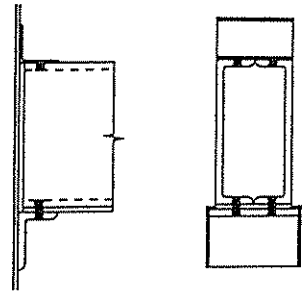


Figure D-13: Typical girder to column connection (System 1, Option 1).

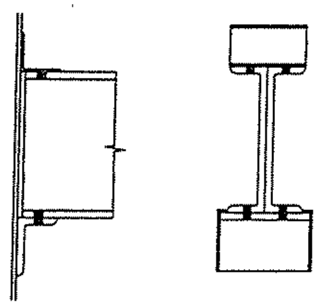


Figure D-14: Typical girder to column connection (System 1, Option 2).

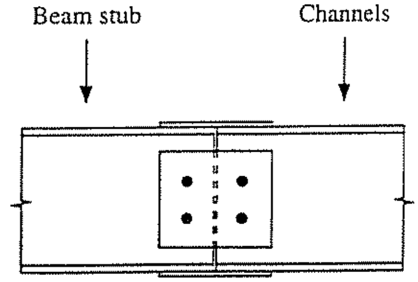


Figure D-15: Typical girder splice. (System 2).

Appendix E: Project Analysis Calculations

In this section we describe the procedure we use to compute the estimated durations and worker air times of the new systems described in Chapter 5, in addition to a traditionally constructed system, for the prototype building described in Appendix D (see Figure D-1). This methodology is based on the site observations discussed in section 4.1, and the procedure used to calculate the air times for Buildings A and B (see Table 4-2), with several modifications. We use a large Quattro Pro Spreadsheet to manage the numerous calculations.

SYSTEM PROPERTIES

The first section of the spreadsheet consists of a summary of the properties of each of the systems, information which is input into the spreadsheet. (All the numbers in italics are input, and the rest are calculated by the spreadsheet.) Table E-1 contains some general information about the buildings, including the number of floors and bays in each building. We divide the bays into three types, Types A, B, and C, depending on their location in the buildings (see Figure E-1). Some of the number and types of members and connections vary depending on the bay type. Table E-2 contains information about the columns in each system, including the number of tiers and the type and number of columns for each system. Table E-3 contains information about the characteristics of each preassembled bay-size panel in each system, depending on the bay type. This table contains information on the number of each type of member and connection in the panels. For System 2, the channel girder to column connections are distinguished by whether the connection is the first (1) or second (2) of the two adjacent panels to be connected to the beam stub. Table E-4 contains information about the number and type of stick-built members in each bay of each system. Table E-5 summarizes the total number of preassembled and stick-built members and calculates the percentage of each. Tables E-6 through E-8 contain connection information for each of the systems, including the number of bolts to be installed and tightened for each connection, and during which stage this will be done (i.e., erection, assembly, or permanent connection). For System 1, the column to column connection involves grouting in addition to the bolts noted in Table E-6. For the innovative column/column/girder connections in System 1, described in Chapter 5, the connection of the panel to upper column is considered to be the column to column connection, the connection of the panel to the lower connection is considered to be the girder to column connection, and for the top story there is an extra panel to column connection associated with connecting the upper part of the panel to the column (since there is no upper column); we used this method of categorization in order to correlate the activities in this system to those of the other two systems.

PRODUCTION RATES FOR SMALL ACTIVITIES

The second section of the spreadsheet consists of a list of production rates for various activities, which are input into the spreadsheet. For these production rates, we used the data in Appendix B ("Average and Interpolated Times of Activities") from the site observations discussed in section 4.1. The production rates of various small activities are listed in Table E-9, according to which major activity they are included in. It should be noted that for production rates for assembly, the setting of the blocks in the approximate locations is neglected; this is because with a series of identical panels, this activity will not have to be repeated every time, and it can initially be done during some of the waiting time of the workers.

DISTRIBUTION OF WORKERS

The next section of the spreadsheet consists of the number of workers that are involved with each activity for each section, and the distribution of how many workers are in the air and on the ground. This information, shown in Tables E-10 through E-12 for the three systems, is used to calculate the distribution of time that is spent by the workers in the air and on the ground. It should be noted that for the activities that take place on completed decking, the workers are considered to be on the ground, because the risk of falling is similar. Some of the activities are listed in Tables E-10 through E-12 with different numbers of total workers involved; this is to allow for the distribution of workers between different activities.

PRODUCTION RATES FOR EACH MAJOR ACTIVITY

From the production rates of the smaller activities described earlier, the production rates of major activities (e.g., Unloading, Shakeout, Assembly, Panel Erection, Stick-Built Erection, Plumbing, Permanent Connection, and Decking) can be calculated. Table E-13 shows the various small activities that we consider for each large activity; those in italics depend on the number of members, connections, or other characteristics of the section and must be calculated for the specific conditions, while the others can be determined readily from the conditions (e.g., some times may depend on the size or the weight of the panel), the production rates of which are shown in Table E-9. Now we describe the results of these calculations for each of the major activities.

Unloading:

For our purposes, the required time for unloading depends on the number of bundles that will be unloaded at a given time, assuming four or five workers are working. We consider a bundle to be a group of 10 members, and we are not distinguishing between the types of members. The production rate for unloading one bundle is given in Table E-9, and the rate for unloading various numbers of bundles is shown in Table E-14.

Shakeout:

As with Unloading, we consider the required time for shakeout to depend on the number of bundles that are being shaken out, assuming four or five workers are working. The production rate for shaking out one bundle is given in Table E-9, and the rate for shaking out various numbers of bundles is shown in Table E-15.

Assembly:

As shown in Table E-13, there are various activities associated with assembling a panel. The smaller activities (those not in italics in Table E-13) can be readily determined from Table E-9. However, we must determine the production rates for the activities that depend on the panel characteristics, including the layout and attachment of individual members, installing the remaining bolts, tightening the bolts, installing the decking sheets, and installing the shear studs, for each type of panel for each system. Examples of these results are shown in Tables E-16 through E-20, respectively.

After calculating the times for the individual activities, we combine these for each of the systems to determine the production rates for Assembly, for different numbers of workers; an example of these results is shown in Table E-21. The three columns in Table E-21 for each bay allow for the calculation of times when some activities occur simultaneously (these activities are marked with a lower-case letter to the left of the activity in the Table E-21). The first column lists the individual time for each activity, the second column compares the simultaneous activities and chooses the highest value, and the third column calculates the cumulative time.

Panel Erection:

As with Assembly, there are various smaller activities associated with erecting a panel (those in Table E-13 that are not in italics) that can be readily determined from Table E-9. However, the times for connecting the panel to the rest of the building depend on other factors. An example of these results is shown in Table E-22.

After finding the times for the individual activities, we combine these for each of the systems to determine the production rates for Panel Erection; examples of these results are shown in Table E-23. As with Assembly, this calculation takes into account the activities that occur simultaneously (these activities are marked with a lower-case letter to the left of the activity in the Table E-23). It should be noted that the production rates are also determined for panels when no preparation time is needed (i.e., attaching the spreader bar to the crane, workers getting into position, and workers climbing down), for the case when more than one panel in a row is erected. We are assuming that seven workers are involved in Panel Erection, including four in the air to make connections (plus one worker operating the crane).

Stick-Built Erection:

As with Assembly and Panel Erection, there are various smaller activities associated with erecting a set of stick-built members (those in Table E-13 that are not in italics) that can be readily determined from Table E-9. However, the times for lifting and attachment of each set of stick-built members to the rest of the building depend on other factors. An example of these results is shown in Table E-24.

After finding the times for the individual activities, we combine these for each of the systems to determine the production rates for Stick-Built Erection, for different sets of members; examples of these results are shown in Table E-25. As with Assembly and Panel Erection, this calculation takes into account the activities that occur simultaneously (these activities are marked with a lower-case letter to the left of the activity in the Table E-25). It should be noted that the production rates are also determined for stick-built sets in some cases when no preparation time is needed (i.e., workers getting into position and workers climbing down), for the case when more than one set of stick-built members in a row is erected. We are assuming that three workers are involved in Stick-Built Erection, including two in the air to make connections (plus one worker operating the crane).

Plumbing:

For our purposes, the required time for plumbing depends on the number of columns that will be plumbed at a given time, assuming two workers are plumbing. The production rate for plumbing one column is given in Table E-9, and the rate for plumbing various numbers of columns is shown in Table E-26.

Permanent Connection:

As with Assembly, Panel Erection, and Stick-Built Erection, there are various smaller activities associated with permanently connecting erected members and panels (those in Table E-13 that are not in italics) that can be readily determined from Table E-9. However, the times for installing the remaining bolts and tightening the bolts (and grouting column to column connections for System 1) for each set of connections depend on other factors. Examples of these results are shown in Tables E-27 through E-29.

After finding the times for the individual activities, we combine these for each of the systems to determine the production rates for Permanent Connection, for different sets of connections and different numbers of workers; examples of these results are shown in Table E-30. It should be noted that the production rates are also determined for permanent connection of sets in some cases when no preparation time is needed (i.e., workers getting into position and workers climbing down), for the case when more than one set of connections in a row is being completed.

Decking:

As with many of the other major activities, there are various smaller activities associated with decking (those in Table E-13 that are not in italics) that can be readily determined from Table E-9. However, the times for installing the decking sheets and shear studs for each bay depend on other factors. Examples of these results are shown in Tables E-31 and E-32, respectively.

After finding the times for the individual activities, we combine these for each of the systems to determine the production rates for Decking, for different bays and different numbers of workers; examples of these results are shown in Table E-33. It should be noted that the production rates are also determined for decking of bays in some cases when no preparation time is needed (i.e., workers getting into position and workers climbing down), for the case when more than one bay in a row is being completed.

FLOW OF ACTIVITIES FOR DIFFERENT SYSTEMS

In order to compute the durations for each system, as we will describe in the following section, we first assume a flow of activities for each system. We determine this flow according to worker availability (assuming a crew of seven workers which can be divided into separate tasks), crane availability (assuming one crane), and which activities have precedence over other activities in order for construction to take place most efficiently. A further assumption we make is that there is limited area for panel assembly, so only two panels may be assembled at a time before they are erected. In deriving this flow, we follow a certain pattern of column and panel erection, so too many columns are not erected before the panels are attached. Figures E-2 through E-4 illustrate the assumed flow for each of the systems.

COMBINING MAJOR ACTIVITIES TO COMPUTE DURATIONS

After we calculate the production rates of the major activities and assume a flow of activities, we use another section of the spreadsheet to sequentially list the major activities, along with the required time for each activity, to calculate the project duration and worker air times for each system. We break the crew of workers into two crews, Crews A and B, which work on simultaneous activities, which we consider when we compute the cumulative times. Tables E-34 through E-36 show excerpts from these spreadsheets. In addition, the number of workers in the air and on the ground during each activity (see Tables E-10 through E-12) is multiplied by the time for that activity to compute a total of all the individual workers' air and ground times. We use these values to calculate the percentage of time spent by workers in the air and on the ground for each system. The final results of this project analysis, including the total duration, total worker time in air, percent of time in air, total worker time on ground, and percent of time on ground, for each of the systems, are shown in Table E-37.

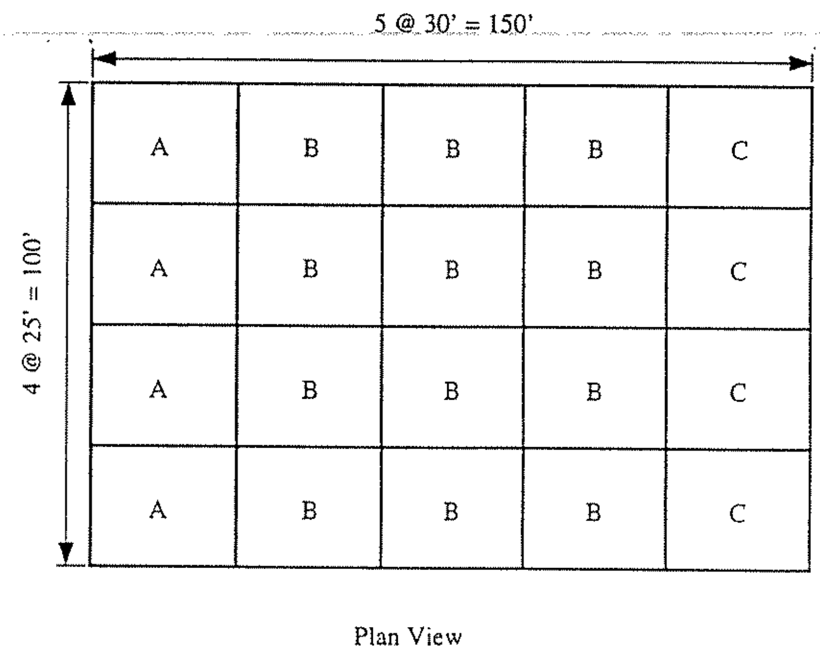


Figure E-1: Illustration of different bay types.

Table E-1: General information.

	System 1	System 2	Traditional
Number of floors	*5*	*5*	*5*
Number of Type A bays per floor	*4*	*4*	*4*
Number of Type B bays per floor	*12*	*12*	*12*
Number of Type C bays per floor	*4*	*4*	*4*
Total number of bays per floor	20	20	20
Total number of bays	100	100	100

Table E-2: Column information.

	System 1	System 2	Traditional
Number of "tiers"	*5*	*3*	*3*
Number of middle tiers	3	1	1
One-story columns/lower tier	*30*	*0*	*0*
Two-story columns/lower tier	*0*	*0*	*30*
Two-story stub columns/lower tier	*0*	*30*	*0*
One-story columns/mid tiers	*30*	*0*	*0*
Two-story columns/mid tiers	*0*	*0*	*30*
Two-story stub columns/mid tiers	*0*	*30*	*0*
One-story columns/upper tier	*30*	*0*	*30*
One-story stub columns/upper tier	*0*	*30*	*0*
Total number of columns	150	90	90

Table E-3: Preassembly information.

Characteristics of each bay-size panel	System 1			System 2			Stick-built		
	Bay A	Bay B	Bay C	Bay A	Bay B	Bay C	Bay A	Bay B	Bay C
Small channel girders (0 conn)	0	0	0	1	2	1	0	0	0
Small W-shape girders (0 conn)	0	0	0	1	0	1	0	0	0
Medium channel girders (0 conn)	1	2	1	0	0	0	0	0	0
Medium W-shape girders (0 conn)	1	0	1	0	0	0	0	0	0
Medium joists (2 conn)	9	9	9	5	5	5	0	0	0
Sheets of decking	10	10	10	10	10	10	0	0	0
Total number of members per panel	21	21	21	17	17	17	0	0	0
Joist to girder connections	18	18	18	10	10	10	0	0	0
Shear studs	165	165	165	105	105	105	0	0	0
Channel girder to column connections (1)	2	4	2	2	2	0	0	0	0
Channel girder to column connections (2)	N/A	N/A	N/A	0	2	2	N/A	N/A	N/A
W-shape girder to column connections	2	0	2	2	0	2	0	0	0
Channel to channel connections	0	12	12	0	7	7	0	0	0

Table E-4: Stick-built information.

Stick-built members and connections in each bay	System 1			System 2			Stick-built		
	Bay A	Bay B	Bay C	Bay A	Bay B	Bay C	Bay A	Bay B	Bay C
Small channel girders (0 conn)	0	0	0	0	0	0	0	0	0
Small W-shape girders (0 conn)	0	0	0	0	0	0	0	0	0
Medium channel girders (0 conn)	0	0	0	0	0	0	0	0	0
Medium W-shape girders (0 conn)	0	0	0	0	0	0	2	1	1
Medium joists (2 conn)	0	0	0	4	4	4	9	9	9
Sheets of decking	0	0	0	0	0	0	10	10	10
Total number of members per panel	0	0	0	4	4	4	21	20	20
Joist to girder connections	0	0	0	8	8	8	18	18	18
Shear stud connections	0	0	0	60	60	60	165	150	150
Channel girder to column connections	0	0	0	0	0	0	0	0	0
W-shape girder to column connections	0	0	0	0	0	0	4	2	2

Table E-5: Summary of total preassembled and stick-built members for each system.

	System 1		System 2		Traditional	
	Preassem.	Stick-built	Preassem.	Stick-built	Preassem.	Stick-built
Small channel girders (0 conn)	0	0	160	0	0	0
Small W-shape girders (0 conn)	0	0	40	0	0	0
Medium channel girders (0 conn)	160	0	0	0	0	0
Medium W-shape girders (0 conn)	40	0	0	0	0	120
Medium joists (2 conn)	900	0	500	400	0	900
Sheets of decking	1000	0	1000	0	0	1000
One-story columns	0	150	0	0	0	30
Two-story columns	0	0	0	0	0	60
Two-story stub columns	0	0	0	30	0	0
Two-story stub columns	0	0	0	60	0	0
TOTAL	2100	150	1700	490	0	2110

	System 1	System 2	Traditional
Percentage preassembled	93%	78%	0%
Percentage stick-built	7%	22%	100%

Table E-6: Connection information for System 1.

Type of connection	Total number of bolts per conn.	Number of bolts to install		Number of bolts to tighten	
		Erection or Assembly	Permanent Connection	Erection or Assembly	Permanent Connection
Column to base	4	4	0	0	4
Column to column*	4	4	0	0	4
Panels to column**	4	0	4	0	4
Joist to girder connection	2	2	0	0	2
Channel girder to column connection	2	2	0	0	2
W-shape girder to column connection	4	2	2	0	4
Channel to channel	2	0	2	0	2

* This connection also involves grouting.

**This is for the top story only, when there is no column to column connection.

Table E-7: Connection information for System 2.

Type of connection	Total number of bolts per conn.	Number of bolts to install		Number of bolts to tighten	
		Erection or Assembly	Permanent Connection	Erection or Assembly	Permanent Connection
Column to base	4	4	0	0	4
Column to column	4	4	0	0	4
Joist to girder connection	2	2	0	0	2
Channel girder to column conn. (1)	2	2	0	0	2
Channel girder to column conn. (2)	6	2	4	0	6
W-shape girder to column connection	8	4	4	0	8
Channel to channel	2	0	2	0	2

Table E-8: Connection information for traditionally constructed system.

Type of connection	Total number of bolts per conn.	Number of bolts to install		Number of bolts to tighten	
		Erection or Assembly	Permanent Connection	Erection or Assembly	Permanent Connection
Column to base	4	4	0	0	4
Column to column	4	4	0	0	4
Joist to girder connection	2	2	0	0	2
W-shape girder to column connection	4	2	2	0	4

Table E-9: Production rates for various activities.

Production rates for miscellaneous activities.

Activity	Time (min)
Unloading each bundle	*2.75*
Shaking out each bundle	*9*
Plumbing each column	*2*

Production rates for assembly.

Activity	Time (min)
Preparing area for medium panel	*3*
Workers getting into position, etc.	*5*
Setting blocks in approximate locations*	*0*
Getting crane into position, etc.	*3*
Layout & attachment of small girders (0 conn)--2 wkrs	*4.25*
Layout & attachment of medium girders (0 conn)--2 wkrs	*5*
Layout & attachment of medium joists (2 conn)--2 wkrs	*2.5*
Final measurement of medium panel	*7*
Installing decking sheets--2 wrks	*3.5*

*Assume that this is only done once in the beginning and is neglected.

Production rates for erection of panels.

Activity	Time (min)
Attaching spreader bar to crane, etc.	*6*
Workers getting into position, etc.	*12*
Attaching spreader bar to & balancing med., light. panel	*6*
Attaching spreader bar to & balancing med., mod. panel	*8*
Lifting medium, light panel into position	*4*
Lifting medium, moderate panel into position	*5*
Aligning medium panel with connections	*5*
Unhooking medium panel	*6*
Move crane back to ground, etc.	*2*
Workers climbing down, etc.	*5*

(continued)

Table E-9 (cont): Production rates for various activities.

Production rates for erection of stick-built members.

Activity	Time (min)
Workers getting into position, etc.--on ground	*5*
Workers getting into position, etc.--in air	*15*
Getting crane into position, etc.	*3*
Lifting & attachment of each one-story column--2 wrks	*6*
Lifting & attachment of each two-story column--2 wrks	*8*
Lifting & attachment of each one-story stub column--2 wrks	*10*
Lifting & attachment of each two-story stub column--2 wrks	*12*
Lifting& attachment of small girders (0 conn)--2 wkrs	*4.75*
Lifting & attachment of medium girders (0 conn)--2 wkrs	*5.5*
Lifting & attachment of medium joists (2 conn)--2 wkrs	*3*
Workers climbing down, etc.--in air	*5*

Production rates for permanent connection (in air).

Activity	Time (min)
Workers getting into position, etc.	*10*
Moving between each connection--1 worker	*1*
Workers climbing down, etc.	*5*

Production rates for decking (in air).

Activity	Time (min)
Workers getting into position, etc.	*10*
Installing decking sheets--2 wrks	*4*
Workers climbing down, etc.	*5*

Production rates for connections made on the ground (each worker).

Activity	Time (min)
Installing each bolt (min)	*0.25*
Tightening each bolt (min)	*3*
Installing each stud (min)	*0.25*

Production rates for connections made in the air (each worker).

Activity	Time (min)
Installing each bolt (min)	*0.5*
Tightening each bolt (min)	*3.5*
Grouting each column splice (min)	*20*
Installing each stud (min)	*0.5*

Table E-10: Distribution of workers for System 1.

Activity	# Workers Total	Air	Ground
Unloading	5	0	5
Shakeout	5	0	5
Assembly	5	0	5
Erection (panel)	7	4	3
Erection (lower col.)	2	0	2
Erection (upper col.)	2	0	2
Plumbing (lower col.)	2	0	2
Plumbing (upper col.)	2	0	2
P.C. (column base)	2	0	2
P.C. (col/col conn's)	2	0	2
P.C. (col/panel conn's)	7	0	7
P.C. (panels)	2	0	2
P.C. (panels)	7	0	7

Table E-11: Distribution of workers for System 2.

Activity	# Workers Total	Air	Ground
Unloading	4	0	4
Unloading	5	0	5
Shakeout	4	0	4
Shakeout	5	0	5
Assembly	4	0	4
Assembly	5	0	5
Erection (panel)	7	4	3
Erection (lower col.)	2	0	2
Erection (upper col.)	2	0	2
Erection (stick-built bays)	3	2	1
Plumbing (lower col.)	2	0	2
Plumbing (upper col.)	2	0	2
P.C. (column base)	2	0	2
P.C. (stick-built bays)	2	2	0
P.C. (stick-built bays)	7	7	0
P.C. (col/col conn's)	2	0	2
P.C. (col/col conn's)	7	0	7
P.C. (panels)	2	0	2
P.C. (panels)	7	0	7
Decking	3	2	1

Table E12: Distribution of workers for traditionally constructed system.

Activity	# Workers Total	Air	Ground
Unloading	4	0	4
Unloading	5	0	5
Shakeout	4	0	4
Shakeout	5	0	5
Erection (lower col.)	2	0	2
Erection (upper col.)	2	0	2
Erection (stick-built bays)	3	2	1
Plumbing (lower col.)	2	0	2
Plumbing (upper col.)	2	0	2
P.C. (column base)	2	0	2
P.C. (column base)	7	0	7
P.C. (col/col conn's)	2	0	2
P.C. (col/col conn's)	7	0	7
P.C. (stick-built bays)	2	2	0
P.C. (stick-built bays)	4	4	0
P.C. (stick-built bays)	5	5	0
P.C. (stick-built bays)	7	7	0
Decking	3	2	1
Decking	7	6	1

Table E-13: Summary of activities.

Major Activity	Smaller Activities
Unloading	None
Shakeout	None
Assembly	•Preparing assembly area •Workers getting into position, etc. •Setting blocks in approximate locations •Getting crane into position, etc. •*Layout and attachment* •Final measurement •*Installing remaining bolts* •*Tightening bolts* •*Installing decking* •*Installing shear studs*
Panel Erection	•Attaching spreader bar to crane •Workers getting into position, etc. •Attaching spreader bar to panel, balancing •Lifting into position •Aligning with connections •*Connecting to building* •Unhooking •Moving crane to ground, etc. •Workers climbing down
Stick-Built Erection	•Workers getting into position •Crane getting into position •*Lifting and attaching one set* •Workers climbing down
Plumbing	None
Permanent Connection	•Workers getting into position, etc. •*Installing remaining bolts* •*Tightening bolts* •*Grouting columns* •Moving between connections •Workers climbing down, etc.
Decking	•Workers getting into position, etc. •*Installing metal decking* •*Installing shear studs* •Workers climbing down

Note: Activities in italics require further calculations to determine a production rate.

Table E-14: Times for Unloading (4-5 workers).

	Number of bundles				
	8	9	10	11	12
Total time to unload (min)	**22**	**25**	**28**	**30**	**33**

Table E-15: Times for Shakeout (4-5 workers).

	Number of bundles				
	8	9	10	11	12
Total time to shakeout (min)	**72**	**81**	**90**	**99**	**108**

Table E-16: Times for layout and attachment of each panel (part of Assembly).

Example: Layout and attachment of each panel in System 1.

Type of member	time/ member	Bay A # memb	Bay A time (min)	Bay B # memb	Bay B time (min)	Bay C # memb	Bay C time (min)
Small girders (0 conn)	4.25	0	0	0	0	0	0
Medium girders (0 conn)	5	2	10	2	10	2	10
Medium joists (2 conn)	2.5	9	22.5	9	22.5	9	22.5
Time for layout--2 workers (min)			**33**		**33**		**33**
Time for layout--4 workers (min)			**16**		**16**		**16**
Time for layout--6 workers (min)			**11**		**11**		**11**

Table E-17: Times for installing remaining bolts for each panel (part of Assembly).

Example: Installing remaining bolts for each panel in System 1.

Type of connection	# bolts to install	Bay A # conn	Bay A time (min)	Bay B # conn	Bay B time (min)	Bay C # conn	Bay C time (min)
Joist to girder	0	18	0	18	0	18	0
Time to install--1 worker (min)			**0**		**0**		**0**
Time to install--2 workers (min)			**0**		**0**		**0**
Time to install--3 workers (min)			**0**		**0**		**0**
Time to install--4 workers (min)			**0**		**0**		**0**
Time to install--5 workers (min)			**0**		**0**		**0**
Time to install--6 workers (min)			**0**		**0**		**0**

Table E-18: Times for tightening bolts for each panel (part of Assembly).

Example: Tightening bolts for each panel in System 1.

		Bay A		Bay B		Bay C	
Type of connection	# bolts to tighten	# conn	time (min)	# conn	time (min)	# conn	time (min)
Joist to girder	2	18	108	18	108	18	108
Time to tighten--1 worker (min)			**108**		**108**		**108**
Time to tighten--2 workers (min)			**54**		**54**		**54**
Time to tighten--3 workers (min)			**36**		**36**		**36**
Time to tighten--4 workers (min)			**27**		**27**		**27**
Time to tighten--5 workers (min)			**22**		**22**		**22**
Time to tighten--6 workers (min)			**18**		**18**		**18**

Table E-19: Times for installing decking sheets for each panel (part of Assembly).

Example: Installing decking sheets for each panel in System 1.

		Bay A		Bay B		Bay C	
Type of member	time/ sheet	# sheets	time (min)	# sheets	time (min)	# sheets	time (min)
Sheets of decking	3.5	10	35	10	35	10	35
Time for installing--2 workers (min)			**35**		**35**		**35**
Time for installing--4 workers (min)			**18**		**18**		**18**
Time for installing--6 workers (min)			**12**		**12**		**12**

Table E-20: Times for installing shear studs for each panel (part of Assembly).

Example: Installing shear studs for each panel in System 1.

Type of connection	Bay A		Bay B		Bay C	
	# studs	time (min)	# studs	time (min)	# studs	time (min)
Shear studs	165	41	165	41	165	41
Time to install--1 worker (min)		**41**		**41**		**41**
Time to install--2 workers (min)		**21**		**21**		**21**
Time to install--3 workers (min)		**14**		**14**		**14**
Time to install--4 workers (min)		**10**		**10**		**10**
Time to install--5 workers (min)		**8**		**8**		**8**
Time to install--6 workers (min)		**7**		**7**		**7**
Time to install--7 workers (min)		**6**		**6**		**6**

Table E-21: Times for Assembly.

Example: Assembly of each panel in System 1 (5 workers).

		Bay A			Bay B			Bay C		
a	Preparing area (min)	3			3			3		
a	Workers getting into position, etc. (min)	5			5			5		
a	Setting blocks in approx. locations (min)	0			0			0		
a	Getting crane into position, etc. (min)	3	5	5	3	5	5	3	5	5
	Layout and attachment (min)	16	16	21	16	16	21	16	16	21
	Final measurement (min)	7	7	28	7	7	28	7	7	28
	Installing remaining bolts (min)	0	0	28	0	0	28	0	0	28
	Tightening all bolts (min)	22	22	50	22	22	50	22	22	50
	Installing decking (min)	18	18	67	18	18	67	18	18	67
	Installing shear studs (min)	8	8	**76**	8	8	**76**	8	8	**76**

Table E-22: Times for connecting each panel to building (part of Panel Erection).

Example: Connecting each panel to building in System 1.

Type of connection	# bolts/ connection	Bay A # conn	Bay A time (min)	Bay B # conn	Bay B time (min)	Bay C # conn	Bay C time (min)
Channel girder to column	2	2	2	4	4	2	2
W-shape girder to column	2	2	2	0	0	2	2
Time for connecting--1 wkr (min)			**4**		**4**		**4**
Time for connecting--2 wkrs (min)			**2**		**2**		**2**
Time for connecting--3 wkrs (min)			**1.33**		**1.33**		**1.33**
Time for connecting--4 wkrs (min)			**1**		**1**		**1**

Table E-23: Times for Panel Erection.

Example: Erection of each panel in System 1 (7 workers).

		Bay A			Bay B			Bay C		
b	Attaching spreader bar, etc. (min)	6			6			6		
b	Workers getting into position, etc. (min)	12	12	12	12	12	12	12	12	12
	Attaching spreader bar to panel, balancing (min)	8	8	20	8	8	20	8	8	20
	Lifting into position (min)	5	5	25	5	5	25	5	5	25
	Aligning with connections (min)	5	5	30	5	5	30	5	5	30
	Connecting to building* (min)	1	1	31	1	1	31	1	1	31
	Unhooking (min)	6	6	37	6	6	37	6	6	37
c	Move crane to ground, etc. (min)	2			2			2		
c	Workers climbing down (min)	5	5	**42**	5	5	**42**	5	5	**42**

* Assuming four workers are connecting.

Example: Erection of each panel in System 1 (7 workers)--no preparation time.

		Bay A			Bay B			Bay C		
	Attaching spreader bar to panel, balancing (min)	8	8	8	8	8	8	8	8	8
	Lifting into position (min)	5	5	13	5	5	13	5	5	13
	Aligning with connections (min)	5	5	18	5	5	18	5	5	18
	Connecting to building* (min)	1	1	19	1	1	19	1	1	19
	Unhooking (min)	6	6	25	6	6	25	6	6	25
	Move crane to ground, etc. (min)	2	2	**27**	2	2	**27**	2	2	**27**

* Assuming four workers are connecting.

Table E-24: Times for lifting and attachment of each set of stick-built members (part of Stick-Built Erection).

Example: Lifting and attachment of each set of stick-built members in System 2.

		Bay A		Bay B		Bay C	
Type of member	**time/ member**	**# memb**	**time (min)**	**# memb**	**time (min)**	**# memb**	**time (min)**
Small girders (0 conn)	4.75	0	0	0	0	0	0
Medium girders (0 conn)	5.5	0	0	0	0	0	0
Medium joists (2 conn)	3	4	12	4	12	4	12
Time for lifting--3 workers* (min)			**12**		**12**		**12**

*Assume two workers are in the air, and one is on the ground.

Table E-25: Times for Stick-Built Erection.

Example: Erection of each set of lower tier columns in System 1 (2 workers).

		4 columns			5 columns			6 columns		
d	Workers getting into position, etc. (min)	5			5			5		
d	Crane getting into position, etc. (min)	3	5	5	3	5	5	3	5	5
	Lifting and attaching one set (min)	24	24	**29**	30	30	**35**	36	36	**41**

Example: Erection of each set of middle tier columns in System 1 (2 workers).

		4 columns			5 columns			6 columns		
e	Workers getting into position, etc. (min)	15			15			15		
e	Crane getting into position, etc. (min)	3	15	15	3	15	15	3	15	15
	Lifting and attaching one set (min)	24	24	39	30	30	45	36	36	51
	Workers climbing down, etc. (min)	5	5	**44**	5	5	**50**	5	5	**56**

Example: Erection of each bay of stick-built members in System 2 (3 workers).

		Bay A			Bay B			Bay C		
f	Workers getting into position, etc. (min)	15			15			15		
f	Crane getting into position, etc. (min)	3	15	15	3	15	15	3	15	15
	Lifting and attaching one set (min)	12	12	27	12	12	27	12	12	27
	Workers climbing down, etc. (min)	5	5	**32**	5	5	**32**	5	5	**32**

Example: Erection of each bay of stick-built members in System 2 (3 workers)--no preparation time.

		Bay A			Bay B			Bay C		
	Lifting and attaching one set (min)	12	12	**12**	12	12	**12**	12	12	**12**

Table E-26: Times for Plumbing (2 workers).

	Num. of columns		
	4	5	6
Total time to plumb (min)	**8**	**10**	**12**

Table E-27: Times for installing remaining bolts (part of Permanent Connection).

Example: Installing remaining bolts for each lower tier column to foundation connection.

	System 1		System 2		Traditional	
Type of connection	**# bolts/ connection**	**time (min)**	**# bolts/ connection**	**time (min)**	**# bolts/ connection**	**time (min)**
Column to base	0	0	0	0	0	0
Time to install--1 worker (min)		**0**		**0**		**0**
Time to install--2 workers (min)		**0**		**0**		**0**
Time to install--3 workers (min)		**0**		**0**		**0**
Time to install--4 workers (min)		**0**		**0**		**0**
Time to install--5 workers (min)		**0**		**0**		**0**
Time to install--6 workers (min)		**0**		**0**		**0**
Time to install--7 workers (min)		**0**		**0**		**0**

Example: Installing remaining bolts for each bay of stick-built members in traditionally constructed system.

		Bay A		Bay B		Bay C	
Type of connection	**# bolts/ connection**	**# conn**	**time (min)**	**# conn**	**time (min)**	**# conn**	**time (min)**
W-shape girder to column	2	4	4	2	2	2	2
Joist to girder	0	18	0	18	0	18	0
Time to install--1 worker (min)			**4**		**2**		**2**
Time to install--2 workers (min)			**2**		**1**		**1**
Time to install--3 workers (min)			**1.33**		**0.67**		**0.67**
Time to install--4 workers (min)			**1**		**0.5**		**0.5**
Time to install--5 workers (min)			**0.8**		**0.4**		**0.4**
Time to install--6 workers (min)			**0.67**		**0.33**		**0.33**
Time to install--7 workers (min)			**0.57**		**0.29**		**0.29**

Table E-28: Times for tightening bolts (part of Permanent Connection).

Example: Tightening bolts for each lower tier column to foundation connection.

Type of connection	System 1		System 2		Traditional	
	# bolts/ connection	time (min)	# bolts/ connection	time (min)	# bolts/ connection	time (min)
Column to base	4	14	4	14	4	14
Time to tighten--1 worker (min)		**14**		**14**		**14**
Time to tighten--2 workers (min)		**7**		**7**		**7**
Time to tighten--3 workers (min)		**5**		**5**		**5**
Time to tighten--4 workers (min)		**4**		**4**		**4**
Time to tighten--5 workers (min)		**2.8**		**2.8**		**2.8**
Time to tighten--6 workers (min)		**2.3**		**2.3**		**2.3**
Time to tighten--7 workers (min)		**2.0**		**2.0**		**2.0**

Example: Tightening bolts for each bay of stick-built members in traditionally constructed system.

Type of connection	# bolts/ connection	Bay A		Bay B		Bay C	
		# conn	time (min)	# conn	time (min)	# conn	time (min)
W-shape girder to column	4	4	56	2	28	2	28
Joist to girder	2	18	126	18	126	18	126
Time to tighten--1 worker (min)			**182**		**154**		**154**
Time to tighten--2 workers (min)			**91**		**77**		**77**
Time to tighten--3 workers (min)			**61**		**51**		**51**
Time to tighten--4 workers (min)			**46**		**39**		**39**
Time to tighten--5 workers (min)			**36**		**31**		**31**
Time to tighten--6 workers (min)			**30**		**26**		**26**
Time to tighten--7 workers (min)			**26**		**22**		**22**

Table E-29: Times for grouting column to column connections (part of Permanent Connection).

Grouting each upper tier column in System 1.

Type of connection	time (min)
Column to column	20
Time to grout--1 worker (min)	**20**
Time to grout--2 workers (min)	**10**
Time to grout--3 workers (min)	**7**
Time to grout--4 workers (min)	**5**

Table E-30: Times for Permanent Connection.

Example: P. C. of each set of lower tier column to foundation connections in System 1 (2 workers).

	4 columns			5 columns			6 columns		
Installing remaining bolts (min)	0	0	0	0	0	0	0	0	0
Tightening bolts (min)	28	28	**28**	35	35	**35**	42	42	**42**

Example: P. C. of each set of column to column connections in System 1 (2 wkrs).

	4 columns			5 columns			6 columns		
Workers getting into position, etc. (min)	10	10	10	10	10	10	10	10	10
Installing remaining bolts (min)	0	0	10	0	0	10	0	0	10
Tightening bolts (min)	28	28	38	35	35	45	42	42	52
Grouting columns (min)	40	40	78	50	50	95	60	60	112
Moving between connections (min)	1	1	79	2	2	97	2	2	114
Workers climbing down, etc. (min)	5	5	**84**	5	5	**102**	5	5	**119**

Example: P. C. of each bay of stick-built members in traditionally constructed system (2 wkrs).

	Bay A			Bay B			Bay C		
Workers getting into position, etc. (min)	10	10	10	10	10	10	10	10	10
Installing remaining bolts (min)	2	2	12	1	1	11	1	1	11
Tightening bolts (min)	91	91	103	77	77	88	77	77	88
Moving between connections (min)	10	10	113	9	9	97	9	9	97
Workers climbing down (min)	5	5	**118**	5	5	**102**	5	5	**102**

Example: P. C. of each bay of stick-built members in traditionally constructed system (2 wkrs)--no prep. time.

	Bay A			Bay B			Bay C		
Installing remaining bolts (min)	2	2	2	1	1	1	1	1	1
Tightening bolts (min)	91	91	93	77	77	78	77	77	78
Moving between connections (min)	10	10	**103**	9	9	**87**	9	9	**87**

Table E-31: Times for installing decking sheets for each bay (part of Decking).

Example: Installing decking sheets for each bay in traditionally constructed system.

		Bay A		Bay B		Bay C	
Type of member	**time/ sheet**	**# sheets**	**time (min)**	**# sheets**	**time (min)**	**# sheets**	**time (min)**
Sheets of decking	4	10	40	10	40	10	40
Time for installing--3 workers* (min)			**40**		**40**		**40**
Time for installing--5 workers* (min)			**20**		**20**		**20**
Time for installing--7 workers* (min)			**13**		**13**		**13**

*Assume one worker is on the ground.

Table E-32: Times for installing shear studs for each bay (part of Decking).

Example: Installing shear studs for each bay in traditionally constructed system.

	Bay A		Bay B		Bay C	
Type of connection	**# studs**	**time (min)**	**# studs**	**time (min)**	**# studs**	**time (min)**
Shear studs	165	83	150	75	150	75
Time to install--1 worker (min)		**83**		**75**		**75**
Time to install--2 workers (min)		**41**		**38**		**38**
Time to install--3 workers (min)		**28**		**25**		**25**
Time to install--4 workers (min)		**21**		**19**		**19**
Time to install--5 workers (min)		**17**		**15**		**15**
Time to install--6 workers (min)		**14**		**13**		**13**
Time to install--7 workers (min)		**12**		**11**		**11**

Table E-33: Times for Decking.

Example: Decking each bay of traditionally constructed system (3 workers).

	Bay A			Bay B			Bay C		
Workers getting into position, etc. (min)	10	10	10	10	10	10	10	10	10
Installing metal decking (min)	40	40	50	40	40	50	40	40	50
Installing shear studs (min)	28	28	78	25	25	75	25	25	75
Workers climbing down (min)	5	5	**83**	5	5	**80**	5	5	**80**

Example: Decking each bay of traditionally constructed system (3 workers)--no preparation time.

	Bay A			Bay B			Bay C		
Installing metal decking (min)	40	40	40	40	40	40	40	40	40
Installing shear studs (min)	28	28	**68**	25	25	**65**	25	25	**65**

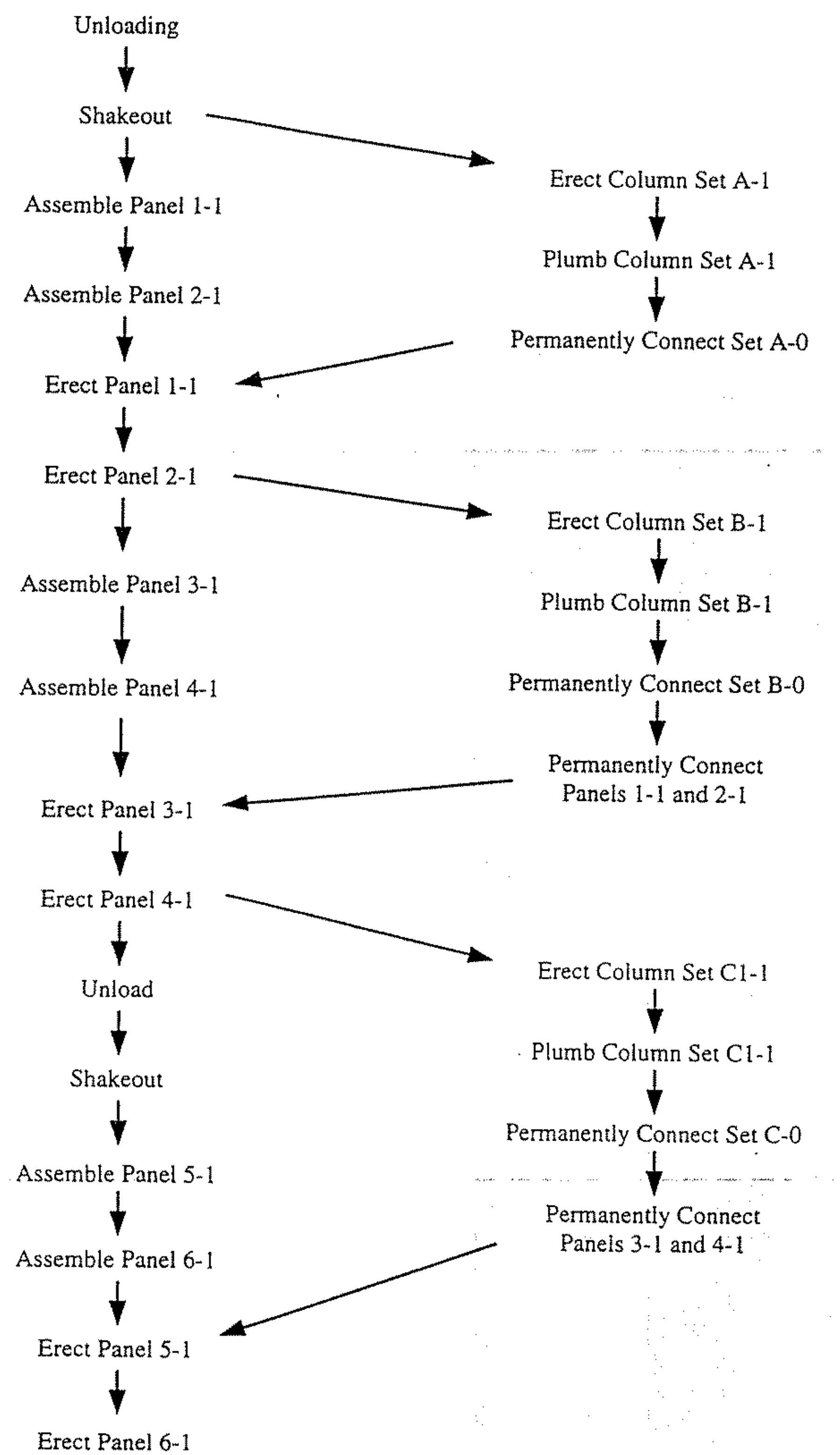


Figure E-2: Sample flow of activities for System 1.

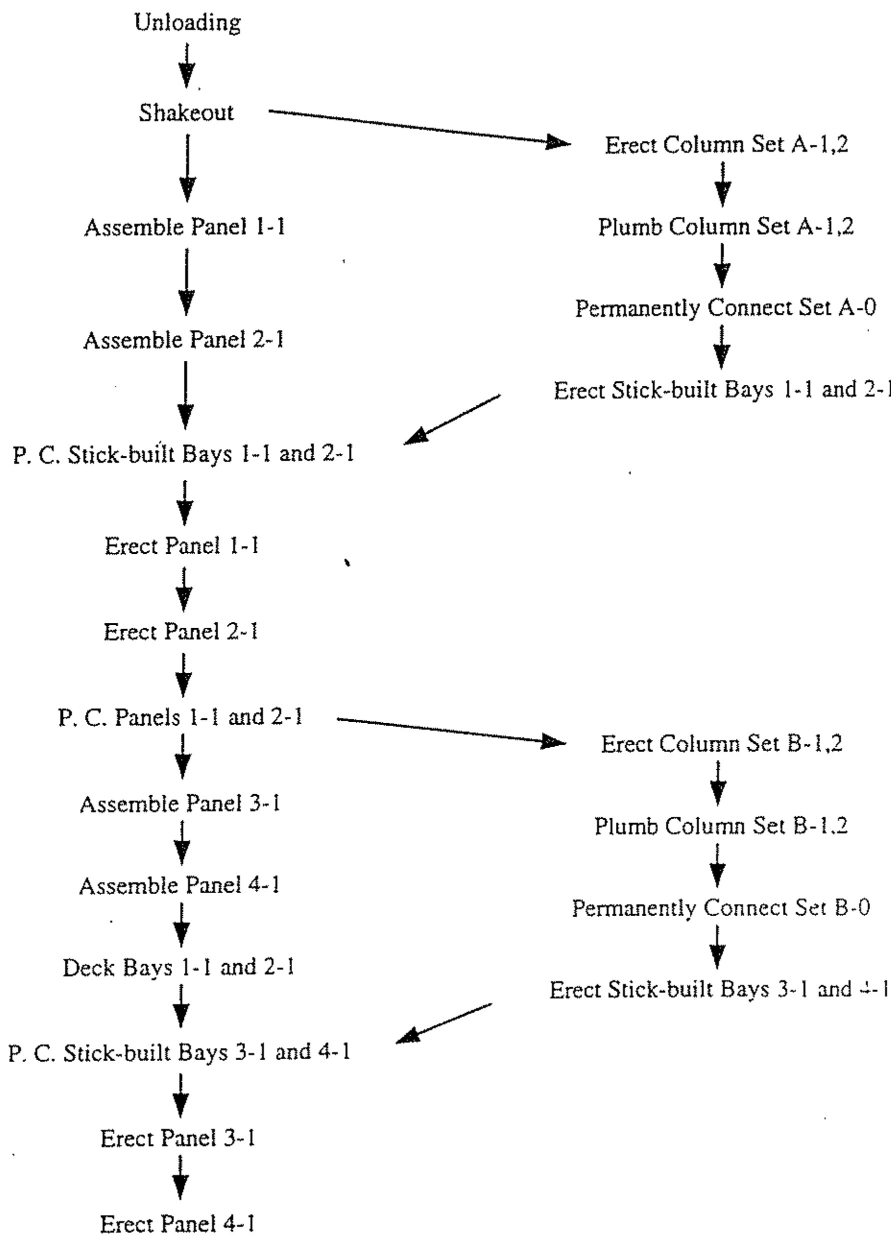


Figure E-3: Sample flow of activities for System 2.

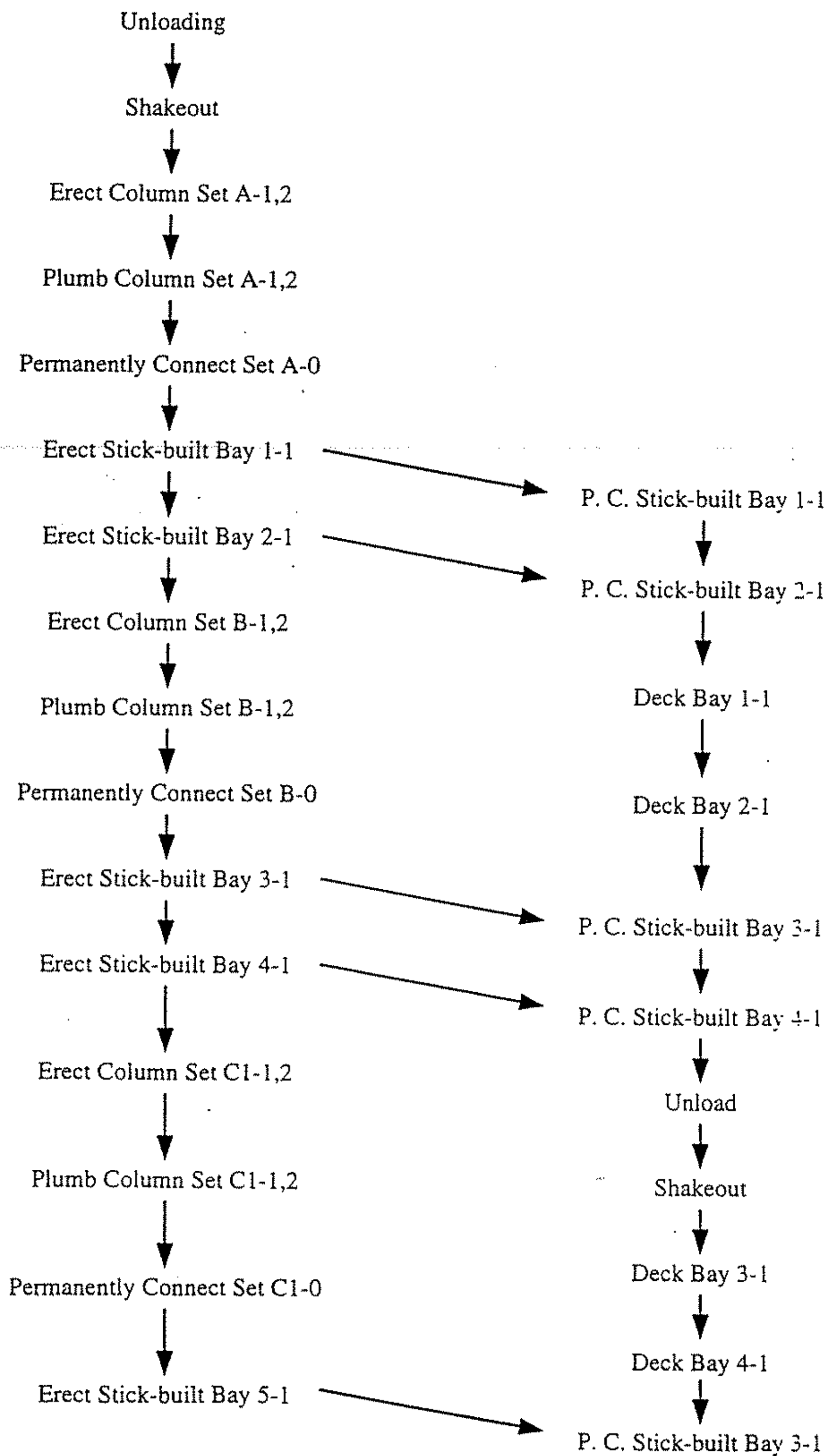


Figure E-4: Sample flow of activities for traditionally constructed system.

Table E-34: Excerpt from duration analysis of System 1.

CREW A					CREW B					DURATION		
			Total Times					Total Times				
Wkrs	Activity	Time (min)	Air (min)	Ground (min)	Wkrs	Activity	Time (min)	Air (min)	Ground (min)	Cumulative Time (min)	(hours)	(days)
					5	Unload 9 bundles	25	0	124	25	0	0.1
					5	Shakeout 9 bundles	81	0	405	106	2	0.2
2	Erect Col set A-1	41	0	82	5	Assemble Panel 1-1	76	0	378	181	3	0.4
2	Plumb Col set A-1	12	0	24	5	Assemble Panel 2-1	76	0	378	257	4	0.5
2	P.C. Set A-0	42	0	84		(Wait for Crew A)	0			257	4	0.5
					7	Erect Panel 1-1	42	168	126	299	5	0.6
					7	Erect Panel 2-1	27	108	81	326	5	0.7
2	Erect Col set B-1	29	0	58	5	Assemble Panel 3-1	76	0	378	402	7	0.8
2	Plumb Col set B-1	8	0	16	5	Assemble Panel 4-1	76	0	378	477	8	1.0
2	P.C. Set B-0	28	0	56								
2	P.C. Panel 1-1	37	0	74								
2	P.C. Panel 2-1	22	0	44		(Wait for Crew A)	0			477	8	1.0
					7	Erect Panel 3-1	42	168	126	519	9	1.1
					7	Erect Panel 4-1	27	108	81	546	9	1.1
2	Erect Col set C1-1	35	0	70	5	Unload 8 bundles	22	0	110	568	9	1.2
2	Plumb Col set C1-1	10	0	20	5	Shakeout 8 bundles	72	0	360	640	11	1.3
2	P.C. Set C1-0	35	0	70	5	Assemble Panel 5-1	76	0	378	716	12	1.5
2	P.C. Panel 3-1	37	0	74	5	Assemble Panel 6-1	76	0	378	791	13	1.6
2	P.C. Panel 4-1	22	0	44		(Wait for Crew A)	0			791	13	1.6
					7	Erect Panel 5-1	42	168	126	833	14	1.7
					7	Erect Panel 6-1	27	108	81	860	14	1.8
2	P.C. Panel 5-1	85	0	170	5	Assemble Panel 7-1	76	0	378	936	16	1.9
2	P.C. Panel 6-1	70	0	140	5	Assemble Panel 8-1	76	0	378	1012	17	2.1
						(Wait for Crew A)	4			1015	17	2.1
					7	Erect Panel 7-1	42	168	126	1057	18	2.2
					7	Erect Panel 8-1	27	108	81	1084	18	2.3

Table E-35: Excerpt from duration analysis of System 2.

CREW A					CREW B					DURATION		
			Total Times					Total Times				
Wkrs	Activity	Time (min)	Air (min)	Ground (min)	Wkrs	Activity	Time (min)	Air (min)	Ground (min)	Cumulative Time (min)	(hours)	(days)
					5	Unload 9 bundles	25	0	124	25	0	0.1
					5	Shakeout 9 bundles	81	0	405	106	2	0.2
2	Erect Col set A-1,2	77	0	154	5	Assemble Panel 1-1	57	0	286	163	3	0.3
2	Plumb Col set A-1,2	12	0	24	5	Assemble Panel 2-1	57	0	286	220	4	0.5
2	P.C. Set A-0	42	0	84						220	4	0.5
3	Erect S.B. Bay 1-1	32	64	32						220	4	0.5
3	Erect S.B. Bay 2-1	12	24	12		(Wait for Crew A)	61			281	5	0.6
					7	P.C. S.B. Bay 1-1	23	162	0	304	5	0.6
					7	P.C. S.B. Bay 2-1	8	57	0	312	5	0.7
					7	Erect Panel 1-1	40	158	119	352	6	0.7
					7	Erect Panel 2-1	25	98	74	376	6	0.8
					7	P.C. Panel 1-1	24	0	165	400	7	0.8
					7	P.C. Panel 2-1	9	0	60	408	7	0.9
2	Erect Col set B-1,2	53	0	106	4	Assemble Panel 3-1	62	0	246	470	8	1.0
2	Plumb Col set B-1,2	8	0	16	4	Assemble Panel 4-1	62	0	246	531	9	1.1
2	P.C. Set B-0	28	0	56	3	Deck Bay 1-1	25	50	25	556	9	1.2
3	Erect S.B. Bay 3-1	32	64	32	3	Deck Bay 2-1	10	20	10	566	9	1.2
3	Erect S.B. Bay 4-1	12	24	12		(Wait for Crew A)	0			566	9	1.2
					7	P.C. S.B. Bay 3-1	23	162	0	589	10	1.2
					7	P.C. S.B. Bay 4-1	8	57	0	598	10	1.2
					7	Erect Panel 3-1	40	158	119	637	11	1.3
					7	Erect Panel 4-1	25	98	74	662	11	1.4
					7	P.C. Panel 3-1	24	0	165	685	11	1.4
					7	P.C. Panel 4-1	9	0	60	694	12	1.4
2	Erect Col set C1-1,2	65	0	130	4	Unload 8 bundles	22	0	88	716	12	1.5
2	Plumb Col set C1-1,2	10	0	20	4	Shakeout 8 bundles	72	0	288	788	13	1.6
2	P.C. Set C1-0	35	0	70	4	Assemble Panel 5-1	62	0	246	849	14	1.8
3	Erect S.B. Bay 5-1	32	64	32	4	Assemble Panel 6-1	62	0	246	911	15	1.9
3	Erect S.B. Bay 6-1	12	24	12						911	15	1.9

Table E-36: Excerpt from duration analysis of traditionally constructed system.

CREW A					CREW B						DURATION		
			Total Times					Total Times					
Wkrs	Activity	Time (min)	Air (min)	Ground (min).	Wkrs	Activity	Time (min)	Air (min)	Ground (min)	Cumulative Time (min)	(hours)	(days)
					5	Unload 9 bundles	25	0	124	25	0	0.1
					5	Shakeout 9 bundles	81	0	405	106	2	0.2
2	Erect Col set A-1,2	53	0	106						106	2	0.2
2	Plumb Col set A-1,2	12	0	24		(Wait for Crew A)	65			171	3	0.4
					7	P.C. Set A-0	12	0	84	183	3	0.4
3	Erect S.B. Bay 1-1	58	116	58		(Wait for Crew A)	58			241	4	0.5
3	Erect S.B. Bay 2-1	38	76	38	4	P.C. S.B. Bay 1-1	66	264	0	307	5	0.6
						(Wait for Crew A)	0			307	5	0.6
2	Erect Col set B-1,2	37	0	74	5	P.C. S.B. Bay 2-1	41	203	0	347	6	0.7
2	Plumb Col set B-1,2	8	0	16	3	Deck Bay 1-1	83	165	83	430	7	0.9
2	P.C. Set B-0	35	0	70	3	Deck Bay 2-1	68	135	68	497	8	1.0
3	Erect S.B. Bay 3-1	58	116	58		(Wait for Crew A)	0			497	8	1.0
3	Erect S.B. Bay 4-1	38	76	38	4	P.C. S.B. Bay 3-1	66	264	0	563	9	1.2
						(Wait for Crew A)	0			563	9	1.2
2	Erect Col set C1-1,2	45	0	90	5	P.C. S.B. Bay 4-1	41	203	0	604	10	1.3
2	Plumb Col set C1-1,2	10	0	20	5	Unload 8 bundles	22	0	110	626	10	1.3
2	P.C. Set C1-0	35	0	70	5	Shakeout 8 bundles	72	0	360	698	12	1.5
3	Erect S.B. Bay 5-1	53	105	53	3	Deck Panel 3-1	83	165	83	780	13	1.6
					3	Deck Panel 4-1	68	135	68	848	14	1.8
						(Wait for Crew A)	0			848	14	1.8
3	Erect S.B. Bay 6-1	33	65	33	4	P.C. S.B. Bay 5-1	58	232	0	906	15	1.9
						(Wait for Crew A)	0			906	15	1.9
3	Erect S.B. Bay 7-1	33	65	33	4	P.C. S.B. Bay 6-1	43	172	0	949	16	2.0
						(Wait for Crew A)	0			949	16	2.0
3	Erect S.B. Bay 8-1	33	65	33	4	P.C. S.B. Bay 7-1	43	172	0	992	17	2.1
						(Wait for Crew A)	0	0	110	992	17	2.1
3	Deck Bay 5-1	80	160	80	4	P.C. S.B. Bay 8-1	51	204	0	1043	17	2.2
3	Deck Bay 6-1	65	130	65	4	Unload 8 bundles	22	0	88	1065	18	2.2
2	Erect Col set C2-1,2	45	0	90	4	Shakeout 8 bundles	72	0	288	1137	19	2.4

Table E-37: Results of Duration Analyses.

	System 1	System 2	Traditional
Total duration (days)	30.7	32.7	24.3
Worker time in air* (days)	28.8	61.1	88.1
Percent of time in air	14%	30%	63%
Worker time on ground* (days)	176.6	144.4	52.3
Percent of time on ground	86%	70%	37%

*This accounts for all the individual worker air times combined, which is why it is greater than the total duration.