New floor framing systems for precast concrete office buildings

Willem J. van Zyverden
Lehigh University

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TITLE: New Floor Framing Systems for Precast Concrete Office Buildings

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NEW FLOOR FRAMING SYSTEMS FOR PRECAST CONCRETE
OFFICE BUILDINGS

by

Willem J. van Zyverden

A Thesis
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in
Civil Engineering

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This thesis is accepted and approved in partial fulfillment of the requirements for the Master of Science.

Aug 5, 1997
Date

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Acknowledgements

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Abstract

This report presents research progress on an ATLSS project entitled, *Development of New Floor Framing Systems for Gravity Loads*. This project has two broad objectives: (1) to explore the development of new structural floor systems for gravity loads; and (2) to develop a methodology for the systematic comparison and evaluation of different structural floor systems. The present work focuses on the development of new precast concrete floor framing systems for office buildings with regular spacing of columns or bearing walls.

Presented are five concepts for new or improved precast concrete floor framing systems. Information provided for each system includes a description of the structural components, the design objective, the anticipated advantages and disadvantages and the approximate structural dimensions of each component for a typical bay dimension of 9.14 m x 9.14 m (30 ft x 30 ft). Also provided for each system is a description of how the HVAC, plumbing, and electrical systems are incorporated, a list of some of the issues which still need to be addressed, and some of the possible variations of the system.

Also presented in some detail is a new precast concrete stub girder, which is an integral component of three of the five systems, and a design approach for the basic shear and flexural design of this girder, using a computer spreadsheet design aid. Also presented are the design specifications for a shored and unshored version of the stub girder used in the system concept which was determined to be the most promising by a panel of industry advisors.
Chapter 1

Introduction

1.1 INTRODUCTION

Structural floor systems comprise a major portion of both the cost and weight of precast concrete building frames. Structural floor systems in multi-story buildings also have a significant impact on the overall building height and on the design and installation of service systems, including plumbing (i.e., supply, drain-waste-vent, and fire protection), HVAC (i.e., heating, ventilation, and air conditioning), and electrical (i.e., power, security, and communication) systems. Therefore, it appears that significant improvements in overall building system efficiency can be gained by improving precast structural floor systems to reduce weight, depth, and cost, and to better accommodate service systems.

Traditional approaches to improving floor system efficiency have sought to minimize the weight, depth, and cost of structural floor systems through the use of new or higher strength materials and improved construction techniques. Automated fabrication and erection technologies offer additional potential for reduced fabrication and erection cost and time. In addition, a third opportunity exists in the better accommodation of service systems. In order to address these opportunities, new and innovative concepts for precast concrete floor systems are needed. The main objective of the research described in this report is to develop new concepts for precast concrete floor systems for gravity loads for application in office buildings with a regular spacing of columns or bearing walls.
1.2 RESEARCH OBJECTIVES

The research presented in this report is part of a project titled *Development of New Floor Framing Systems for Gravity Loads*. The project has two broad objectives:

1. To develop new structural systems for gravity loads; and,
2. To develop a methodology for the systematic comparison and evaluation of different structural floor systems.

The present work is focused on the development of new precast concrete floor systems for gravity loads for application in office buildings with regular spacing of columns or bearing walls. The project evolved from a Precast/Prestressed Concrete Institute (PCI) research project statement entitled "Economical Framing Systems for Floors and Roofs." It was expanded to include some of the broader research interests of the authors, and to consider the broader strategic objectives of the ATLSS Center.

1.3 RESEARCH SIGNIFICANCE

The research addresses the need for innovation in the construction of precast concrete buildings. Structural floor systems are a major portion of both the cost and weight of precast concrete buildings. Structural floor systems also have a significant impact on the overall height of multi-story building, and on the design and installation of service systems. The new precast concrete structural systems sought by this research offer the potential for significant improvements in overall building efficiency.
1.4 SUMMARY OF APPROACH

The research has been separated into three tasks as follows:

**Task 1: Assess current and emerging precast concrete structural floor systems**
- Task 1.1 - survey of current and emerging precast structural systems;
- Task 1.2 - develop assessment criteria to evaluate precast structural systems;
- Task 1.3 - apply assessment criteria to precast structural systems in the survey.

**Task 2: Develop new concepts for precast structural floor systems**
- Task 2.1 - identify opportunities for new precast structural systems;
- Task 2.2 - propose new concepts for precast structural floor systems;
- Task 2.3 - apply assessment criteria to new precast structural systems.

**Task 3: Detailed development of most promising new concepts from Task 2**

The results of the work performed up to and including Tasks 2.1 were presented in an ATLSS report titled "Identification and Preliminary Assessment of Existing Precast Concrete Floor Framing Systems" [Prior, Pessiki, Sause, Slaughter, and van Zyverden 1993], and a Master's Thesis of the same name [Prior 1993].

This report presents the results of Task 2.2 - propose new concepts for precast structural systems, and Task 3 - Detailed development of most promising new concepts from Task 2. The proposed concepts presented in this report incorporate what was learned from Tasks 1.1 through 2.1.
1.5 SUMMARY OF FINDINGS

The effort to develop new or improved precast concrete floor framing systems resulted in the development of five systems, and the development of a new type girder which is referred to as the stub girder. This girder is similar in concept to the stub girder developed for steel construction [Colaco 1972]. The stub girder offers the advantage of providing a means of passing the building services through the girder rather than under it, leading to a net reduction in the total floor system depth.

System 1 uses double tee floor members in conjunction with the stub girder. System 2 uses a precast channel member with corrugated steel deck that acts as a stay-in-place form for a cast-in-place slab which spans between adjacent channel members. Precast planks may also be used in place of corrugated steel deck with this system. The girders supporting the channels are standard inverted tees. System 3 is similar to System 2, but uses the stub girder in place of the standard inverted tee. System 4 uses prestressed joists, supported by standard inverted tees, with precast planks placed between adjacent joists. The precast planks serve as stay-in-place forms for a cast-in-place topping slab which acts compositely with the joists. Corrugated steel deck may also be used in place of the precast planks for this system. System 5 is the same as the fourth system, but uses the stub girder in place of the standard inverted tee.

Each system offers certain key advantages for particular design needs. System 1, by means of the double tee floor members, offers the capability for long spans, and provides a significant amount of floor area with the placement of each member. The relatively large openings provided in the stub girder enable the horizontal passage of services through the
girder rather than below it, which helps to minimize the total floor depth. System 2, with the ability to vary the span of the cast-in-place slab between channel members, offers advantages for accommodating irregular or varying building geometries. The space between channels can provide a convenient location for openings which enable the vertical passage of services through the floor system. This space can also be used for the placement of some of the larger components of the HVAC system, such as diffusers and variable-air-volume (VAV) boxes. System 3 offers all the same advantages of System 2, but with the additional advantages provided by the use of the stub girder which can help to minimize the total floor depth. System 4, with the ability to vary the joist spacing, is able to achieve relatively long spans with minimum floor depth. This variability of joist spacing also provides flexibility for accommodating irregular or varying building geometries. Requiring minimum prefabrication and being more field-labor oriented, this system also has the potential of providing certain cost-saving advantages in areas where low-cost field labor is available. System 5 offers all the same advantages of System 4, but with the additional advantage of minimal total floor depth provided by the use of the stub girder.

1.6 SCOPE OF REPORT

Chapter 2 summarizes the work conducted in the first phase of this research project, which serves as the basis for the development of the new concepts presented in Chapter 4.

Chapter 3 describes the methodology followed in the development of the proposed concepts presented in Chapter 4.

Chapter 4 presents five proposed new concepts for precast structural floor framing
systems. Information presented for each system includes the design objective of the system, and a discussion of its potential advantages and disadvantages. Structural dimensions are given for a typical bay size, but these dimensions are approximate, based on preliminary calculations only.

Chapter 5 presents a design for the stub girder used for System 1 which was determined to be the most promising of the system concepts of Chapter 4. The information includes a description of the design process and a summary of the calculated design specifications for a bay size of 9.75 m x 12.19 m (32 ft x 40 ft) for both shored and unshored construction.

Chapter 6 discusses alternate configurations for System 1, which includes possibilities for moment resistant connections for the stub girder, and the use of a variety of different bay dimensions. A table summarizes some of the design specifications of System 1 required for selected combinations of floor member and girder lengths.

Chapter 7 summarizes the report and discusses future research needs.

Appendix A provides a complete example of the calculations carried out for the analysis and design of an unshored stub girder. The calculations were performed using a computer spreadsheet design aid which was developed specifically for the design of the stub girder. A step-by-step design procedure using the spreadsheet design aid is also provided.

1.7 TERMINOLOGY

The following terminology is used throughout this report:

*Building System* - The structural, service, and architectural systems of the building;

*Structural System* - All structural components of the building, including precast members,
cast-in-place members, cast-in-place connections, welded connections, and bolted connections;

**Service Systems** - Electrical (i.e., power, security, and communication) system, plumbing (i.e. supply, drain-waste-vent, and fire protection) system, and HVAC equipment and ductwork;

**Architectural System** - Architectural elements including interior spaces, building function, materials, partitions, exterior enclosures, noise control, thermal storage, and safety system;

**Floor System** - The structural floor system and the space below the floor (and above the finished ceiling of the level below) that is required for service systems and architectural systems;

**Structural Floor System** - The combined structural components which make up the floor system (i.e. girders, beams, floor members, cast-in-place concrete slab etc.);

**Total Floor Depth** - The total depth of the floor system, measured from the finished ceiling below the floor system to the top of the finished floor above the floor system.
Chapter 2

Background

2.1 INTRODUCTION

This chapter summarizes the results of the work conducted in the first phase of this research project (Tasks 1.1 - 1.3, 2.1). This first phase focused on the identification and evaluation of existing precast concrete floor framing systems, and is presented in detail in an ATLSS report titled "Identification and Preliminary Assessment of Existing Precast Concrete Floor Framing Systems" by Prior et al. [1993] and papers by Pessiki et al. [1994a, 1994b].

2.2 OPPORTUNITIES FOR NEW PRECAST STRUCTURAL SYSTEMS

The work by Prior et al. [1993] identified opportunities for the development of new precast structural systems. The opportunities were presented in terms of desired physical attributes of the structural system which lead to improved structural, service, and architectural efficiency and performance.

It was found that in some cases the desired physical attributes of the structural system are directly linked to improved structural, service, and architectural efficiency and performance. In other cases, the desired physical attributes of the structural system are linked to desired physical attributes of the service or architectural systems, and these service or architectural system attributes lead to improved efficiency and performance.

The impact of a particular physical attribute was presented in terms of selected criteria
that are described fully in Prior et al. [1993]. It was found that many of the physical attributes impact more than one efficiency and/or performance criterion. In some cases, contradictions arise whereby a change in a particular physical attribute may cause improved efficiency and/or performance in terms of some criteria, but reduced efficiency and/or performance in terms of other criteria.

2.2.1 Structural Efficiency and Performance

Prior et al. [1993] identified a number of physical attributes of the precast structural system that are linked directly to improved structural efficiency and performance. The desired changes in these physical attributes are listed in Table 2.1, along with the efficiency and performance criteria impacted by these attributes. To summarize, Prior et al. [1993] presented the following opportunities to improve structural efficiency and performance of precast structural systems:

1. Reduce the number of pieces, reduce the number of different pieces, use pieces which can be mass produced, and increase modularity to improve the efficiency of structural fabrication.

2. Use smaller precast pieces, which group together efficiently on a truck, to improve transportation efficiency.

3. Reduce the number of precast pieces, reduce the quantity of field-placed reinforcement, reduce the amount of form work for cast-in-place concrete, reduce the number of connections, and simplify connections to allow for quick erection
to improve the efficiency of erection operations.

4. Increase member strength, connection strength, and member stiffness to improve structural performance in terms of overall strength and deflections.

2.2.2 Service Efficiency and Performance

Prior et al. [1993] also identified several physical attributes of the precast structural system that can impact service efficiency and performance, as summarized in Table 2.2. As presented by Prior et al., opportunities to improve service efficiency and performance of precast structural systems include:

1. Reduce the depth of the structural system, avoid the integration of services in restrictive openings in the structural system to increase space for services, and therefore to improve service design, installation, capacity and versatility.

2. Increase the space between ribs in ribbed structural systems to increase space for services and thereby improve efficiency of service maintenance and modification.

3. Avoid the integration of services in restrictive openings in the structural system to increase space for services, and therefore to improve the efficiency of service maintenance and modification.

4. Reduce structural depth and integrate services in the structural system to reduce building volume by reducing total floor depth, and therefore to increase the operation efficiency of heating, ventilating and air conditioning service systems.

5. Reduce structural depth and use large horizontal openings that easily accommodate service systems to increase space for services, and therefore to
improve the performance of the service systems in terms of capacity and versatility.

2.2.3 Architectural Efficiency and Performance

Finally, Prior et al. [1993] identified a number of physical attributes of the structural system that can impact architectural efficiency and performance, as shown in Table 2.3. The criteria considered in group A involve only the precast structural system. For the criteria considered in group B, the desired physical attributes of the structural system are linked to desired physical attributes of the architectural systems, and these architectural system attributes lead to improved architectural efficiency and performance. To summarize, Prior et al. presented the following opportunities to improve architectural efficiency and performance of precast structural systems:

1. Increase modularity in the structural system to improve the efficiency of architectural design.
2. Provide the ability to accommodate large vertical openings in the structural system to improve the efficiency of architectural modification.
3. Provide the ability to accommodate large vertical openings, a range of span lengths, and non-rectilinear floor plans to improve the architectural performance in terms of spatial and functional versatility.
4. Reduce structural depth and integrate services in the structural system to improve the efficient use of architectural materials and improve the performance in terms of building height versatility.
Objective: Improve structural efficiency and performance

<table>
<thead>
<tr>
<th>Criteria Considered</th>
<th>Approach (Structural System)</th>
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<tbody>
<tr>
<td><strong>Efficiency</strong></td>
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<tr>
<td>Fabrication Operations</td>
<td>Reduce number of precast pieces</td>
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<tr>
<td></td>
<td>Reduce number of different precast pieces</td>
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<tr>
<td></td>
<td>Use pieces which can be mass produced</td>
</tr>
<tr>
<td></td>
<td>Increase modularity</td>
</tr>
<tr>
<td>Truck Requirements</td>
<td>Use smaller pieces which group together efficiently</td>
</tr>
<tr>
<td>Erection Operations</td>
<td>Reduce number of precast pieces</td>
</tr>
<tr>
<td></td>
<td>Reduce quantity of field-placed reinforcement</td>
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<td></td>
<td>Reduce form work for cast-in-place concrete</td>
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<tr>
<td></td>
<td>Simplify and reduce the number of connections</td>
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<tr>
<td><strong>Performance</strong></td>
<td></td>
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<tr>
<td>Strength</td>
<td>Increase member strength</td>
</tr>
<tr>
<td></td>
<td>Increase connection strength</td>
</tr>
<tr>
<td>Deflection of Structural Floor System</td>
<td>Increase member stiffness</td>
</tr>
</tbody>
</table>

Table 2.1. Approach to improve structural efficiency and performance [Prior et al.1993].
Objective: Improve service efficiency and performance

<table>
<thead>
<tr>
<th>Criteria Considered</th>
<th>Approach (Service and Architectural System)</th>
<th>Approach (Structural System)</th>
</tr>
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<tbody>
<tr>
<td><strong>Efficiency</strong></td>
<td></td>
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</tr>
<tr>
<td>Service Design and Service Installation</td>
<td>Increase space for services</td>
<td>Reduce structural depth</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Avoid integration of services</td>
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<tr>
<td></td>
<td></td>
<td>Use large horizontal openings</td>
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<tr>
<td>Service Maintenance and Service Modification</td>
<td>Increase space for services</td>
<td>Increase space between tee stems</td>
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<td></td>
<td>Avoid encased or restricted access</td>
<td>Use large horizontal openings</td>
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<td>Avoid integration of services</td>
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<tr>
<td>Service Operation</td>
<td>Reduce building volume by reducing total floor depth</td>
<td>Reduce structural depth</td>
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<tr>
<td></td>
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<td>Integrate services</td>
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<tr>
<td><strong>Performance</strong></td>
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<tr>
<td>Service Capacity</td>
<td>Increase space for services</td>
<td>Reduce structural depth</td>
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<td>Use large horizontal openings</td>
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<tr>
<td>Service Versatility</td>
<td>Increase space for services</td>
<td>Reduce structural depth</td>
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<td></td>
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<td>Use large horizontal openings</td>
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</tbody>
</table>

Table 2.2. Approach to improve service efficiency and performance [Prior et al. 1993].
**Objective:** Improve architectural efficiency and performance

<table>
<thead>
<tr>
<th>Criteria Considered (Group A)</th>
<th>Approach (Structural System)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Efficiency</td>
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<tr>
<td>Architectural Design</td>
<td>Increase modularity</td>
</tr>
<tr>
<td>Architectural Modification</td>
<td>Accommodate large vertical openings</td>
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<tr>
<td>Performance</td>
<td></td>
</tr>
<tr>
<td>Spatial and Functional Versatility</td>
<td>Accommodate a range of span lengths</td>
</tr>
<tr>
<td></td>
<td>Accommodate non-rectilinear spaces</td>
</tr>
<tr>
<td></td>
<td>Accommodate large vertical openings</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Criteria Considered (Group B)</th>
<th>Approach (Service and Architectural System)</th>
<th>(Structural System)</th>
</tr>
</thead>
<tbody>
<tr>
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<td></td>
<td></td>
</tr>
<tr>
<td>Architectural Construction - materials</td>
<td>Reduce total floor depth</td>
<td>Reduce structural depth</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Integrate services</td>
</tr>
<tr>
<td>Performance</td>
<td></td>
<td></td>
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<tr>
<td>Building Height Versatility</td>
<td>Reduce total floor depth</td>
<td>Reduce structural depth</td>
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<tr>
<td></td>
<td></td>
<td>Integrate services</td>
</tr>
</tbody>
</table>

Table 2.3. Approach to improve architectural efficiency and performance [Prior et al. 1993]
Chapter 3

Methodology

3.1 INTRODUCTION

This chapter describes the methodology used for the development of new precast concrete structural floor framing systems. Earlier work by Prior et al. [1993] focused on the identification and evaluation of existing precast concrete floor framing systems, and identified opportunities for the development of new precast structural systems as summarized in Chapter 2 of this report. This phase of research used what was learned from the work of Prior et al. [1993] as the foundation for the development of a series of preliminary concepts. These preliminary concepts were then presented to a selected group of industry professionals for review. The preliminary concepts that were perceived as the most promising by the industry professionals were then revised according to their comments and suggestions. The resulting five concepts are presented in Chapter 4. These five concepts were then reviewed a second time, in their revised form, to determine which of the five held the most potential for improvement of current precast floor framing methods. Finally a detailed analysis and design was carried out for the most promising of the five proposed systems of Chapter 4.

3.2 CLASSIFICATION OF FLOOR SYSTEMS

After re-examining the existing systems identified by Prior et al. [1993] and the conclusions made about opportunities for developing new precast structural systems, several
observations were made.

The first observation was that it would not be possible to address all the opportunities with any one system. In fact, as noted by Prior et al. [1993], the fulfillment of some opportunities automatically excludes the possible fulfillment of certain other opportunities. For example, integrating the services with the structural system by means of openings in the structural members may achieve the goal of reducing the depth of the total floor system, but this integration may also inhibit efficient service maintenance, or reduce the service capacity.

Another observation was that, while some systems may rate poorly in terms of the number of pieces or the amount of field work required, such as shoring or formwork, this does not mean that these systems are necessarily less effective than other systems which do not possess these characteristics. It was found that the effectiveness of one system over another is often influenced by factors such as geographic location or the cost and availability of labor. For example, in areas where labor is less expensive and the weather conditions are generally favorable for on-site work, it may be economical to perform more of the fabrication on site, rather than in the precast plant. Transportation costs do not vary greatly from region to region. Instead, the impact of transportation costs on the cost effectiveness of one system over another is directly related to the proximity of the precast plant to the building site. If the distance between the precast plant and the building site is large, it may be economical to increase the amount of cast-in-place concrete and decrease the size and weight of the precast components that must be shipped to the site.

Based on the foregoing, it was found that most floor systems could be generally categorized by two particular characteristics. The first characteristic is the manner in which the structural system accommodates the service systems:
Layered Systems - Floor systems in which the structural system and service systems occupy separate layers within the total floor depth. In such a system, the objective is to minimize the thickness of the structural system, thus decreasing the total floor system depth.

Integrated Systems - Floor systems in which the structural system and the service systems are integrated with one another such that the services are essentially contained within the structural depth of the floor system. In such a system, openings are placed in the structural system to allow for the passage of the service systems within the structural depth of the floor system. The objective here is to provide a structural system with adequate depth to accommodate the service systems for the purpose of achieving a decrease in the total floor system depth.

The second characteristic that can be used to classify floor systems is the relative amounts of field work and prefabrication work required by a system:

Field Oriented Systems - Floor systems for which the majority of the fabrication is performed on site. Such systems typically have a large number of lightweight precast components, and require a large amount of formwork, cast-in-place concrete, and temporary shoring. A system of this type may be a preferable choice in areas where labor costs are low and weather conditions are generally
favorable for on-site work.

**Prefabrication Oriented Systems** - Floor systems for which the majority of the fabrication is conducted off site at a precast plant. Such systems typically have a small number of large precast components, requiring minimal formwork and cast-in-place concrete. A system of this type may be a preferable choice in areas where local labor is expensive, but transportation costs are not significant.

The forgoing leads to the following four general categories of floor systems:

1. Layered, field oriented systems
2. Layered, prefabrication oriented systems
3. Integrated, field oriented systems
4. Integrated, prefabrication oriented systems

While any particular system (existing or proposed) may not fall clearly into one particular category, the above categorization does provide a convenient frame of reference to view existing and proposed systems.

It was decided that a comprehensive approach for the development of a preliminary set of concepts would attempt to cover these four classes of floor systems in addition to addressing the opportunities defined by Prior et al. [1993].
3.3 APPROACH TO DEVELOP NEW CONCEPTS

The approach taken to develop new precast floor framing systems consisted of three steps as outlined below.

Step 1 - Preliminary concepts:

The objective of this step was to assemble a number of preliminary concepts that would each address one or more of the opportunities defined by Prior et al. [1993], and would cover the four classes of systems defined in Section 3.2. To begin this process, the existing systems identified by Prior et al. [1993] were studied in detail to gain an understanding of how certain advantages are achieved by each system and what shortcomings each system may have. Using this information as a guide, a number of preliminary floor system concepts were devised.

Step 2 - Industry review:

In this step, descriptions of the preliminary floor system concepts developed in Step 1 were assembled into a document which summarized the concepts. The summaries included information such as the anticipated spanning capabilities and the anticipated advantages and disadvantages. This summary document was then sent out for review by industry professionals consisting mainly of precast concrete fabricators and designers. The reviewers were:

- Kenneth C. Baur, Director of Engineering & Project Management, High Concrete Structures, Inc.
These reviewers were subsequently interviewed to obtain their comments on the preliminary floor system concepts. Generally the comments focused on the concept behind each preliminary system. Discussion of the structural feasibility of each system was limited by the lack of detailed development of the systems, but comments about some possible structural concerns were made.

Step 3 - Revision of preliminary concepts:

The preliminary floor system concepts were studied to obtain approximate dimensions for the individual components to achieve a suitable bay size. It was generally agreed by the industry professionals that a 9.14 m x 9.14 m (30 ft x 30 ft) would be a good target bay size for office structures. It was beyond the scope of this report to develop methods of design and analysis for certain complex features of some of the systems, but certain components of each floor system were designed according to the ACI code.
Chapter 4

System Descriptions

4.1 INTRODUCTION

This chapter presents five precast concrete structural floor system concepts. The feasibility and structural dimensions for these five systems were estimated by preliminary analysis. Each of the proposed floor system concepts are described using the same six part format as follows: 1) **Overview** describes the different components of the system and how they fit together. 2) **Design objectives / potential advantages and disadvantages** describes the intended objectives of the system and the potential advantage and disadvantages of the system. 3) **Incorporation of services** describes how the structural system accommodates the service systems. 4) **Anticipated structural dimensions for a typical bay** gives certain anticipated dimensions of each component of the system for a bay size of about 9.14 m x 9.14 m (30 ft x 30 ft), based on preliminary calculations. The total floor depth was estimated based on the anticipated member dimensions and the assumption that the service systems could be contained within a depth of 102 mm (14 in). The assumed depth of 102 mm (14 in) is intended to be adequate for locations in the building where the service systems include nothing larger than the secondary portions of the HVAC system. 5) **Outstanding issues** indicates particular technical aspects of the system which need to be addressed or explored in more detail. 6) **System variations** presents other member combinations which were considered for the given system, and the potential advantages and disadvantages of these other combinations.
4.2 SYSTEM 1 - Double tees with stub girder

Overview:

System 1 consists of precast stub girders and precast double tee floor members. A perspective view of the assembled system is shown in Figure 4.1. This system is similar to the conventional double tee floor system with a few key differences. The first key difference is the replacement of the inverted tee girder with a precast stub girder which is similar to a steel stub girder [Colaco 1972]. The stub girder used for this system is shown in Figure 4.2. This stub girder is similar in cross section to a standard inverted tee, but has a deeper and wider bottom flange with gaps provided at the top of the member which are about 406 to 457 mm deep (16 in to 18 in) and 762 to 1219 mm wide (30 in to 48 in). The actual size, number and location of these gaps is a function of the particular application. The stub girder relies on composite action for its required flexural strength, and although it is not a requirement, shoring may be used during construction to reduce the required girder depth. The second key difference is a slightly modified double tee which has the first 610 mm (24 in) of its top flange removed from either end, exposing the tops of each stem. This modified double tee is shown in Figure 4.3.

System 1 is classified as an Integrated, prefabrication oriented system according to the classification defined in Chapter 3, although, a fully integrated version of the system would have openings through the stems of the double tees for the passage of services.

To provide the required compression flange, the cast-in-place slab must be approximately 102 to 152 mm (4 to 6 in) deep over the stub girder. A 102 or 152 mm (4 or
6 in) slab over the entire floor system, when a slab of that depth is required only at the location of the girders, is undesirable. Thus the first 610 mm (24 in) of the top flange of the double tees are removed, to enable a thin topping slab to be placed over the entire floor and a deep compression flange to be placed at the location of girders. Details for casting the flange remain to be worked out. As one possible approach, a stay-in-place plywood form may be placed across the tops of the exposed stems of the double tees and between the stubs for the cast-in-place topping. If the removal of the top flange of the double tees still does not provide an adequately deep compression flange, the tops of the double tee stems may be coped to achieve the necessary depth. A cross section of the double tee and stub girder connection with coped stems is shown in Figure 4.4.

**Design objectives / potential advantages and disadvantages:**

The objective of this system is to decrease the total depth of the double tee floor member/inverted tee girder system by replacing the inverted tee with a stub girder. The stub girder will allow the passage of services across the girder line within the structural depth of the floor system through the openings provided in the stub girder.

The issue of girder depth and the passage of services across the girder line is a common problem with many precast floor systems. As a result, considerable effort was devoted to developing feasible solutions to this problem. There are many ways to improve the span to depth ratio of a girder. One way is to use post-tensioning to transform a series of single span members into one continuous member. This is effective, but much of the industry input on this matter indicated that it would be desirable to avoid post-tensioning.
Another approach is to use depressed strands. This is also effective, but it was found that straight strands are preferred by industry. Utilizing composite action with shoring for the non-composite stages of construction is another effective approach, but shoring increases the amount of field work. The precast stub girder, which makes a good substitute for the inverted tee, was found to have the most potential to allow services to pass across the girder line with a reasonable total floor depth. This member uses straight strands, and could be cast in much the same way as the conventional inverted tee. Preliminary analysis, however, has indicated that debonding of a portion of the strands furthest from the neutral axis at the ends of the member may be required in order to achieve the minimum possible depth. To further reduce the depth it may be desirable to use shoring during construction, but shoring is not a requirement.

It is realized that in order to be able to provide an adequate opening for the services, the stub girder depth would most likely be larger than a comparable inverted tee. Preliminary calculations indicate, however, that it is feasible to provide adequate openings and still achieve a reduction in total floor system depth.

The integration of services with the floor system is an advantage in terms of reducing the total floor system depth, but a potential disadvantage in terms of installation and maintenance of the services. As discussed in Prior et al. [1993], structural systems which require the services to be threaded through openings of limited size make service installation difficult, and also provide reduced access for service maintenance.

A key problem with the stub girder is the depth of the bottom flange that is required for unshored construction. The bottom flange of a conventional inverted tee is usually only
about 152 mm (6 in) deep which leaves the remaining depth of the member available to accommodate the depth of the members framing into it. In some cases minimum dapping of the members framing into the inverted tee is required. The stub girder, in its current form, has a significantly larger bottom flange, thus larger dapping of the members framing into it may be required to achieve minimum girder depth. However, this dapping of ends is undesirable and may lead to strand placement and shear capacity problems.

Incorporation of services:

As is common with the conventional double tee floor system, the service systems running longitudinally can be suspended between the stems of the double tees, and the service systems running transverse to the double tees, they can be suspended below the stems. The stub girder enables services that must cross the girder line, that normally would have to pass under the girder, to now pass through it. This offers the potential advantage of reducing the total depth of the floor system. Services pass vertically through openings in the floor system that are either cut or cast in the top flange of the double tees. The size and placement of these openings is limited, however, by the location of the stems of the double tees.

Structural dimensions for a typical bay:

For a 9.75 m x 9 m (32 ft x 30 ft) bay size with 2.44 m (8 ft) wide double tees, the depth for the double tee is 457 mm (18 in), and the depth of the stub girder, using three 813 mm x 457 mm (32 in x 18 in) gaps, is 914 mm (36 in), unshored.

Accounting for a 102 mm (4 in) cast-in-place compression flange over the stub
girders, a girder depth of 914 mm (36 in), and a 13 mm (1/2 in) neoprene bearing pad, the total depth of the floor system is 1029 mm (40 1/2 in). This depth assumes that the openings provided for the services would be adequate to contain all the services within the structural depth of the floor system.

Outstanding issues:

- Shear transfer in the stub girder in the region where the cast-in-place slab connects with the top of the stubs, and where the stub connects with the continuous portion of the girder;
- The reduction in the required dimensions of the stub girder that could be achieved by using shored construction;
- Details for the reinforcing of the cast-in-place slab where it acts as the compression zone for the stub girder;
- Methods of forming the cast-in-place concrete in the region of the stub girder;
- Use of moment resisting connections for the stub girder to reduce the required depth.

System Variations:

One variation of this system is to include openings in the stems of the double tees for the passage of services. This would allow even greater integration of the service systems within the structural system.
Figure 4.1. Perspective view of System 1.
Figure 4.2. Stub girder used for System 1.

Figure 4.3. Modified double tee for System 1.
Figure 4.4. Cross section of double tee and stub girder connection.
4.3 SYSTEM 2 - Cast-in-place slab on channel members with inverted tee girders

Overview:

System 2 is composed of inverted tee girders supporting 1.52 m (5 ft) wide channel members spaced at a maximum of about 4.57 m (15 ft) on centers. The channel members in turn support corrugated steel decking that acts as a stay-in-place form for a cast-in-place slab that is approximately 102 mm (4 in) thick. A perspective view of the assembled system is shown in Figure 4.5. The channel member is similar, but slightly wider between stems than the standard 2.44 m (8 ft) double tee. A typical channel member is shown in Figure 4.6. The cast-in-place slab is continuous over the entire floor, but the recessed ledges in the channel members, which support the steel deck, allow for a deeper slab between channel members where it carries the floor loads. A cross-section of the channel member and cast-in-place slab connection is shown in Figure 4.7. System 2 is classified as a Layered, prefabrication oriented system according to the classification defined in Chapter 2.

Design objectives / potential advantages and disadvantages:

The objective of this system is to utilize the advantages of the conventional double tee floor system and eliminate some of its disadvantages. The conventional double tee is an efficient section with excellent spanning capabilities. It is stable immediately after placement, and for many applications the top flange can serve directly as the finished floor surface.

One of the disadvantages of the double tee, however, is its limited ability to accommodate irregular or varying building geometries. The maximum and minimum center-
to-center spacing of the double tees is limited by the dimensions of the double tee. If necessary, the widths of the exterior flanges can be decreased by altering the forms, but this option has limitations. A channel member retains many of the advantages of the double tee, but replacing the exterior flanges of the double tee with a reinforced cast-in-place slab enables almost any desired member spacing to be accommodated. For office building applications it is standard practice to provide a topping slab for leveling purposes when using double tees. It is anticipated that the additional time and material required to modify the topping slab into a reinforced slab that distributes load between the channels will not be significant.

A key element to the success of this system is the type of stay-in-place form used to support the cast-in-place slab. Ideally the form must be lightweight, easy to install, and able to support the weight of the cast-in-place slab for spans up to about 3.05 m (10 ft). Without a form that can fulfill these criteria, it is likely that many of the advantages of this system over the conventional double tee system will be lost. The details of this issue remain to be solved, but corrugated steel deck is one possibly effective option, due mainly to its lightweight and ease of handling. Precast planks may also be a viable option, but more study is required in this area.

Another advantage offered by System 2 is the accommodation of services passing vertically through the floor system. With the standard double tee, openings can be provided in the top flange during casting or to created on site by cutting or boring, but their locations are limited by the stems. With the channel system, the area between the stems of adjacent members is likely to be larger thus providing fewer limitations on the placement of openings in the floor. Because the channel, like the double tee, is stable immediately after placement,
the top flange of the channel provides a safe and convenient working surface for the placement of forms for the cast-in-place slab.

**Incorporation of services:**

The parts of the service systems which must run parallel to the girder line, between channel members, could be fastened directly to the underside of the cast-in-place slab. At these locations, a region almost equivalent to the entire depth of the girder would be available for the service systems. This large space would allow placement of some of the larger portions of the HVAC system, such as diffusers and variable-air-volume (VAV) boxes. At the locations where the services pass transverse to the channel members, the services will have to pass under the stems. The services which cross the girder line will have to pass under the inverted tees and this will add to the total depth of the floor system. Vertical passage of services could be accomplished by openings provided in either the top flange of the channel members or the cast-in-place slab that spans between them. Depending on the spacing of the channel members, this system may present fewer constraints on the size and location of services passing vertically through the floor system than the conventional double tee system.

**Anticipated structural dimensions for a typical bay:**

For a 9 m x 9 m (30 ft x 30 ft) bay size, a channel spacing of 4.57 m (15 ft) on centers and steel deck for the stay-in-place form, the channel depth is about 559 mm (22 in). The inverted tee girder, using a 51 mm (2 in) dap in the channels, is 813 mm (32 in) deep. For channel members spaced at 4.57 m (15 ft) on centers the span of the cast-in-place slab is 3.05
m (10 ft). To achieve this with steel deck a 102 mm (4 in) slab is required.

Accounting for 51 mm (2 in) of the cast-in-place slab which would cover the top of the girders, and assuming the services passing under the girders will only require an additional 356 mm (14 in) of space, the total floor system depth is 1219 mm (48 in).

Outstanding issues:

- A comparison of this system for a given span length and a given spacing of the channel members to a similar floor system made up of double tees in terms of structural depth, total floor depth and weight per square foot of floor area;
- The advantages and disadvantages of using precast planks to support the cast-in-place slab versus steel deck or some equivalent;
- How to sufficiently anchor the steel deck to the channel members to prevent it from moving in a strong wind before placement of the concrete.

System variations:

Two possible variations of this system were considered for this report. The first variation incorporated the use of precast planks in place of the steel deck. A perspective view of this variation is shown in Figure 4.8. Precast planks may result in a larger tributary load on the channel members. This issue, however, needs to be looked at in more detail. Precast planks also may not be as economical to install as steel deck since they will be placed with the aid of a crane.

The other possible variation is one that incorporates openings in the stems of the
channel members for the passage of services within the structural depth of the channel members. This would not allow services to pass across the girder line without adding to the total floor depth, but this option may still be quite useful in certain circumstances. Channel members with a large center-to-center spacing will become quite deep, and the passage of services under their stems could increase the total floor system depth.
Figure 4.5. Perspective view of System 2.
Figure 4.6. Typical channel member for System 2.

Figure 4.7. Cross-section of the channel member and cast-in-place slab.
Figure 4.8. Variation of System 2 using precast planks in place of steel deck.
4.4 SYSTEM 3 - Cast-in-place slab on channel members with stub girders

Overview:

System 3 utilizes the same channel and slab construction as System 2, but replaces the inverted tee girder with a stub girder similar to the one used in System 1. Figure 4.9 shows a perspective view of the assembled system. The stub girder used for this system is shown in Figure 4.10. This stub girder differs from the stub girder shown for System 1 (Figure 4.1) in the size and location of the gaps, between stubs. The size and location of these gaps depends on the spacing of the channel members and structural requirements for the particular application. The channel members differ only slightly from those used in System 2 in that, like the double tees used for System 1, the first 610 mm (24 in) of the top flange is removed from either end, exposing the tops of each stem. This facilitates the placement of a cast-in-place compression flange for the stub girder. Figure 4.11 shows a typical channel member for this system. Figure 4.12 shows a cross-section of a typical channel member and stub girder connection. This system is classified as an Integrated, prefabrication oriented system according to the classification defined in Chapter 2.

Design objectives / potential advantages and disadvantages:

The objective of this system is to reduce the total depth of System 2 at the girder line while retaining all of its other advantages. For System 2 the services pass under the girder, and this controls the total floor depth. The objective is to use the stub girder in place of the inverted tee to obtain the advantage of integrating the services within the structural depth of the floor system.
As mentioned for System 1, use of the stub girder to integrate the services with the floor system has the advantage of potentially reducing total floor system depth. However, this integration may also be a disadvantage from the perspective of installation and maintenance of the service systems.

**Incorporation of services:**

This system would accommodate the service systems in the same manner as System 2, but with the added feature of being able to pass services through the gaps in the stub girders rather than under the girders. With this advantage it may be possible to contain all the services within the structural depth of the floor system.

**Structural dimensions for a typical bay:**

For a 9 m x 9 m bay size (30 ft x 30 ft), a channel spacing of 4.57 m (15 ft) on center and steel deck for the stay-in-place form, the channel depth is 559 mm (22 in). The depth of the precast stub girder, incorporating two 914 mm x 356 mm (36 in x 14 in) openings for services, is 914 mm (36 in). For this girder depth, a 102 mm (4 in) dap in the ends of the channel members is required. With channel members spaced at 4.57 m (15 ft) on centers the cast-in-place slab span is 3.05 m (10 ft), and, to achieve this with steel deck, a 102 mm (4 in) slab is required.

Accounting for bearing pads and the addition of a 102 mm (4 in) slab to the top of the 914 mm (36 in) deep stub girder, the total depth of the floor system is 1029 mm (40 1/2 in). This depth assumes that all the services are contained within the structural depth of the floor system.
Outstanding issues:

- Shear transfer in the stub girder in the region where the cast-in-place slab connects with the top of the stubs, and where the stub connects with the continuous portion of the girder;
- The reduction in the required dimensions of the stub girder that could be achieved by means of shored construction;
- Details for the required reinforcing in the cast-in-place slab where it acts as the compression zone for the stub girder.

System variations:

One variation of this system, similar to the variation considered for System 2, incorporates the use of a precast planks in place of steel deck for the stay-in-place form. A perspective view of this variation is shown in Figure 4.13 Other possible variations of this system include channel members with openings provided in the stems for the passage of the services. This would allow even greater integration of the service systems within the structural system.
Figure 4.9. Perspective view of System 3.
Figure 4.10. Stub girder used for System 3.

Figure 4.11. Typical channel member for System 3.
Figure 4.12. Cross-section of channel member and stub girder connection.
Figure 4.13. Variation of System 3 using precast planks in place of steel deck.
4.5 SYSTEM 4 - Prestressed joists with a cast-in-place slab and inverted tee girders

Overview:

System 4 is an adaptation of the Prestressed Joist systems by Cast-Crete of Tampa, and Prestressed Systems Industries of Miami (PSI). The system consists of inverted tees supporting prestressed rectangular joists which in turn support precast planks. The precast planks serve as forms for a cast-in-place composite slab. A perspective view of the assembled system is shown in Figure 4.14. The joists are 165 mm (6 1/2 in) wide with a depth that ranges from about 406 mm to 610 mm (16 in to 24 in) depending on the desired spacing and span length. The joists rely on composite action with a cast-in-place slab to carry the superimposed dead and live loads. To achieve this composite action, wire mesh shear reinforcement is extended through the top of the joist to transfer shear between the joist and the cast-in-place slab that covers the joist. However, the shear transfer between the joist and the composite slab needs further study. This system is classified as a *Layered, field oriented system* according to the classification defined in Chapter 3.

Design objectives / potential advantages and disadvantages:

The objective of this system is to develop a better method of constructing the prestressed joist systems developed by PSI and Cast-Crete. Although labor-intensive, the prestressed joist system produces a shallow, lightweight floor system with good spanning capabilities. The systems, as constructed by PSI and Cast-Crete, however, involves extensive formwork and temporary shoring. The goal was to reduce this on-site work as much as
possible. The first change was the replacement of the forming intensive soffit girder with a conventional inverted tee. The next step was to improve the method of forming the cast-in-place slab. The problem was to provide an easy to install, stay-in-place form that would allow composite action to developed between the joist and cast-in-place slab. The approach taken was to use precast planks that run longitudinally between joists but not continuously over them. The gap between the edges of adjacent planks, which meet over a joist, enables the shear reinforcement protruding from the top of the joist to extend into the cast-in-place slab.

An advantage of the prestressed joist system is that there are few fixed dimensions. This makes it possible to divide a bay size into any desired joist spacing. This flexibility makes it possible to achieve shallow structural depths for any given span. The trade-off, however, is that a narrow spacing of joists needed to achieve the minimum structural depth translates into more field work and a larger number of pieces per square foot of floor area.

A disadvantage of this system is the lack of stability that the joists may have for long spans, prior to sufficient strength gain in the cast-in-place slab. This stability problem may be handled through the use of temporary bracing, but the placement and breakdown of this bracing will add to the amount of field work.

Incorporation of services:

Services that run parallel with the joists could be suspended between the joists. Those services that run perpendicular to the joists have to be suspended below them. Services, that cross the girder line, pass under the girders thus adding to the total depth of the floor system. Services pass vertically through openings in the cast-in-place slab between joists. The size and location of these openings would therefore be affected by the chosen spacing of the joists.
Structural dimensions for a typical bay:

A 9 m x 9 m (30 ft x 30 ft) bay size with a 0.92 m (3 ft) joist spacing requires 406 mm (16 in) deep joists with 51 mm (2 in) thick precast planks to support a 51 mm (2 in) cast-in-place slab. The inverted tee girders which support the joists are 711 mm (28 in) deep.

Accounting for a space of 356 mm (14 in) for the services that pass under the girders, and 102 mm (4 in) for the cast-in-place slab and precast plank that are above the girders, the total floor system depth is 1168 mm (46 in).

Outstanding issues:

- The shear transfer between the joists and composite slab.

System variations:

Only one variation of this system was considered. This variation uses corrugated steel deck in place of precast planks for the stay-in-place form which supports the cast-in-place slab. A perspective view of this variation is shown in Figure 4.15.

The advantage of this variation is that the steel deck is lightweight and easy to handle. One problem which has not been resolved is how to create an adequate seal between the steel deck and the joists so that the cast-in-place concrete does not run out under the flutes in the steel deck. Manufacturers of steel deck make inserts specifically for this type of situation, but such a large number of these inserts are needed that use of the inserts may prove to be impractical. Another problem which has not been resolved is how to attach the decking to the joist so that it does not move in a strong wind before placement of the concrete.
Figure 4.14. Perspective view of System 4.
Figure 4.15. Variation of System 4 using steel deck in place of precast planks.
4.6 SYSTEM 5 - Prestressed joists with cast-in-place slab and precast stub girders

Overview:

System 5 consists of precast stub girders supporting prestressed joists which in turn support precast planks. The precast planks serve as forms for a cast-in-place composite slab. A perspective view of the assembled system is shown in Figure 4.16. System 5 is very similar to System 4, but with a few key changes. The stub girder is used in place of the inverted tee girder, and the first 610 mm (24 in) of both ends of each joist are coped 51 mm (2 in) to allow a deep compression flange to be cast at the location of the stub girders. The stub girder for System 5 differs from the stub girders used for System 1 and 3 in the size and locations of the gaps provided in the member. The size and location of the gaps depends on the structural requirements and the spacing of the joists. Figure 4.17 shows the stub girder used for this system, and Figure 4.18 shows a cross-section of the joist and stub girder connection. This system is classified as an Integrated, field oriented system according to the classification defined in Chapter 3.

Design objectives / potential advantages and disadvantages:

The objective of this system is to minimize the total floor depth of System 4 at the girder line through the use of the stub girder which allows the services to pass within the structural depth of the floor system.

This system has the same advantages and disadvantages of System 4, but, through the use of the stub girder, it offers the additional advantage of reducing the total floor depth at
the girder line. However, as mentioned for Systems 1 and 3, the integration of services with the structural system may be a disadvantage in terms of installation and maintenance of the service systems.

**Incorporation of services:**

This system accommodates the service systems in the same manner as System 4 but with the added feature of being able to pass services through the girders rather than below them. Thus, it may be possible to place all the services within the structural depth of the floor system.

**Structural dimensions for a typical bay:**

A 9 m x 9 m (30 ft x 30 ft) bay size with a 0.91 m (3 ft) joist spacing requires 406 mm (16 in) deep joists with 51 mm (2 in) thick precast planks to support a 51 mm (2 in) cast-in-place slab. The depth of the precast stub girders which support the joists are 914 mm (36 in).

Accounting for the 102 mm (4 in) portion of the cast-in-place which extends above the girder, the neoprene bearing pad, and the 914 mm (36 in) depth of the girder itself, the total floor system depth is 1029 mm (40 1/2 in). This depth assumes that all the services are placed within the structural depth of the floor system.

**Outstanding issues:**

- Shear transfer in the stub girder where the cast-in-place slab connects with the top of the stubs, and interface between the stub and the continuous portion of the girder;
• The reduction in the required dimensions of the stub girder that could be achieved by means of shored construction;
• Details of the reinforcing of the cast-in-place slab where it acts as the compression zone for the stub girder;
• Methods of forming the cast-in-place concrete in the region of the stub girder;
• Use of moment resisting connections for the stub girder in order to reduce the required depth.
• The shear transfer between the joists and the composite slab.

System variations:

Only one variation has been considered. This variation, similar to the variation for System 4, uses corrugated steel deck in place of precast planks for the stay-in-place form which supports the cast-in-place slab. A perspective view of this variation is shown in Figure 4.19.

As noted earlier for System 4, the advantage of this variation is that the steel deck is lightweight and easy to handle. One problem which has not been resolved is how to create an adequate seal between the steel deck and the joists so that the cast-in-place concrete does not run out under the flutes in the steel deck. Manufacturers of steel deck make inserts specifically for this type of situation, but such a large number of these inserts are needed that this approach may prove to be impractical. Another problem which has not yet been resolved is how to attach the decking to the joist so that it does not move in a strong wind before placement of the concrete.
Figure 4.16. Perspective view of System 5.
Figure 4.17. Stub girder used for System 5.

Figure 4.18. Cross-section of prestressed joist and stub girder connection.
Figure 4.19. Variation of System 5 using steel deck in place of precast planks.
4.7 OPPORTUNITIES ADDRESSED BY THE PROPOSED FLOOR SYSTEMS

As presented in Chapter 2, the work of Prior et al. [1993] identified opportunities for the development of new precast structural floor systems. The five systems presented in this chapter each address one or more of these opportunities. Tables 4.1, 4.2, and 4.3 summarize the opportunities which each system addresses in terms of their physical attributes.

As determined by Prior et al. [1993], these physical attributes are in many cases directly linked to improved structural, service, and architectural efficiency and performance. In other cases the desired physical attributes of the structural system are linked to desired physical attributes of the service or architectural systems, and these service or architectural system attributes lead to improved efficiency and performance.

The impact of a particular physical attribute was presented in terms of selected criteria that are described fully in Prior et al. [1993]. It was found that many of the physical attributes impact more than one efficiency and/or performance criterion. In some cases, contradictions arise whereby a change in a particular physical attribute may cause improved efficiency and/or performance in terms of some criteria, but reduced efficiency and/or performance in terms of other criteria.
<table>
<thead>
<tr>
<th>Criteria Considered</th>
<th>Approach (Structural System)</th>
<th>System</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1  2</td>
</tr>
<tr>
<td><strong>Efficiency</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fabrication Operations</td>
<td>Reduce number of precast pieces</td>
<td>✓</td>
</tr>
<tr>
<td></td>
<td>Reduce number of different precast pieces</td>
<td>✓</td>
</tr>
<tr>
<td></td>
<td>Use pieces which can be mass produced</td>
<td>✓</td>
</tr>
<tr>
<td></td>
<td>Increase modularity</td>
<td>✓</td>
</tr>
<tr>
<td>Truck Requirements</td>
<td>Use smaller pieces which group together efficiently</td>
<td></td>
</tr>
<tr>
<td>Erection Operations</td>
<td>Reduce number of precast pieces</td>
<td>✓</td>
</tr>
<tr>
<td></td>
<td>Reduce quantity of field-placed reinforcement</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Reduce formwork for cast-in-place concrete</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Simplify and reduce the number of connections</td>
<td></td>
</tr>
<tr>
<td><strong>Performance</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength</td>
<td>Increase member strength</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Increase connection strength</td>
<td></td>
</tr>
<tr>
<td>Deflection of Floor</td>
<td>Increase Member Stiffness</td>
<td></td>
</tr>
<tr>
<td>System</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.1. Structural efficiency and performance opportunities [Prior et al. 1993] addressed by the proposed systems.
### Objective: Improve service efficiency and performance

<table>
<thead>
<tr>
<th>Criteria Considered</th>
<th>Approach</th>
<th>System</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Service and Architectural System</td>
<td>Structural System</td>
</tr>
<tr>
<td>Efficiency</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service Design and Service</td>
<td>Increase space for services</td>
<td>Reduce structural depth</td>
</tr>
<tr>
<td>Installation</td>
<td></td>
<td>Avoid integration of services</td>
</tr>
<tr>
<td>Service Maintenance and Service</td>
<td>Increase space for services</td>
<td>Increase space between tee stems</td>
</tr>
<tr>
<td>Modification</td>
<td>Increase space for services, Avoid encased or restricted access</td>
<td>Use large horizontal openings</td>
</tr>
<tr>
<td>Service Operation</td>
<td>Reduce building volume by reducing total</td>
<td>Reduce structural depth</td>
</tr>
<tr>
<td></td>
<td>floor depth</td>
<td>Integrate services</td>
</tr>
<tr>
<td>Performance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service Capacity</td>
<td>Increase space for services</td>
<td>Reduce structural depth</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Use large horizontal openings</td>
</tr>
<tr>
<td>Service Versatility</td>
<td>Increase space for services</td>
<td>Reduce structural depth</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Use large horizontal openings</td>
</tr>
</tbody>
</table>

Table 4.2. Service efficiency and performance opportunities [Prior et al. 1993] addressed by the proposed systems.
Objectives: Improve architectural efficiency and performance

<table>
<thead>
<tr>
<th>Criteria Considered (Group A)</th>
<th>Approach (Structural System)</th>
<th>System</th>
</tr>
</thead>
<tbody>
<tr>
<td>Efficiency</td>
<td></td>
<td>1 2 3 4 5</td>
</tr>
<tr>
<td>Architectural Design</td>
<td>Increase modularity</td>
<td></td>
</tr>
<tr>
<td>Architectural Modification</td>
<td>Accommodate large vertical openings</td>
<td>✓ ✓</td>
</tr>
<tr>
<td>Performance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spatial and Functional Versatility</td>
<td>Accommodate a range of span lengths</td>
<td>✓ ✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td></td>
<td>Accommodate non-rectilinear spaces</td>
<td>✓ ✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td></td>
<td>Accommodate large vertical openings</td>
<td>✓ ✓</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Criteria Considered (Group B)</th>
<th>Approach</th>
<th>System</th>
</tr>
</thead>
<tbody>
<tr>
<td>Efficiency</td>
<td></td>
<td>1 2 3 4 5</td>
</tr>
<tr>
<td>Architectural Construction - materials</td>
<td>Reduce total floor depth</td>
<td>Reduce Structural depth Integrate Services ✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td>Performance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Building Height Versatility</td>
<td>Reduce total floor depth</td>
<td>Reduce structural depth Integrate services ✓ ✓ ✓ ✓</td>
</tr>
</tbody>
</table>

Table 4.3. Architectural efficiency and performance opportunities [Prior et al. 1993] addressed by the proposed systems.
Chapter 5

Stub Girder Design

5.1 INTRODUCTION

This chapter presents the shear and flexure design of the stub girder used in System 1. This system consists of double tee floor members supported by stub girders, resting on either multi-story or single-story columns. It was the general consensus of the industry advisors, that, out of the five floor system concepts of Chapter 4, System 1 holds the most promise for improvement of current floor framing methods. This chapter focuses on the analysis and design of the stub girder which is the key component of the system. The design approach proposed for the stub girder used in System 1, however, could be used for other systems which employ the stub girder.

The industry advisors' comments on System 1 included making better use of the spanning capability of the double tee floor members. Hollowcore planks, which typically provide very shallow structural floor depths, can be used quite effectively for office loading for spans up to about 10.36 m (34 ft). Double tees, though deeper than hollowcore planks, have a significantly greater spanning capability. To better exploit this greater spanning capability, without requiring dapping (as suggested by the industry advisors), a 12.19 m (40 ft) double tee span was chosen for the detailed design. Longer spans can also be used. However, if the required depth of the double tee increases beyond about 508 mm (20 in), it becomes necessary to use dapped ends for the double tees in order to accommodate an efficient stub girder design that is not excessively deep.
5.2 DESIGN APPROACH

This section provides a description of the approach taken for the shear and flexure design of the stub girder used in System 1. The design approach does not treat the detailing issues associated with the composite slab or the supplementary reinforcement likely to be needed around the gap regions. As discussed in Chapter 7, more work is still needed in this area.

System 1 involves essentially all the same analysis and design issues associated with the common method of floor construction which uses double tee floor members and inverted tee girders. The main challenge with System 1, however, is the analysis and design of the stub girder. As a result, this section has been devoted to a description of the design approach followed for the stub girder. Appendix A provides a detailed description and an example of the design calculations carried out for the unshored stub girder presented in Section 5.4.1. In the sections that follow, specific reference is made to particular sections of Appendix A.

5.2.1 Design Parameters

All designs use normal weight precast concrete with a 28 day compressive strength of 34.5 MPa (5,000 psi), and normal weight cast-in-place concrete with a 28 day compressive strength of 24.1 MPa (3,500 psi). Release strength for the precast concrete is assumed to be 24.1 MPa (3,500 psi). The concrete strengths specified are not a requirement, but the use of different concrete strengths has not been investigated.

The stub girder is designed on the basis of allowable stresses, and then checked for serviceability and strength requirements. Where applicable, all design calculations are carried
out according to the ACI 318-89 Code [ACI 1989]. For strength calculations, load factors of 1.4 and 1.7 are applied to the dead and live loads respectively. For strength checks, the nominal flexure and shear strengths are reduced by factors of 0.90 and 0.85 respectively. Except for deflections, all calculations to proportion the member are based on a linear elastic, uncracked section for a simply supported member. The tensile stresses at the time of prestress transfer are checked, but are not controlling factors for the design. Supplementary reinforcement is proportioned to accommodate the tensile stresses at those locations where the allowable tensile stresses at transfer are exceeded.

Only gravity loading is considered for the analysis and design of the floor system. The dead load is based on the use of double tees made with normal weight concrete, and a 51 mm (2 in) normal weight cast-in-place topping. Also included is a superimposed dead load of 0.7 kPa (15 psf), and a live load of 2.4 kPa (50 psf). The gravity loads applied to the stub girder consist of a series of concentrated loads at the reaction points for the double tee floor members. It was found reasonable to approximate this series of concentrated loads with an equivalent uniform load.

Seven-wire, 1862 MPa (270 ksi) grade, 13 mm (1/2 in) diameter, low-relaxation strand is used for all designs. All designs also employ a straight strand pattern with 76 mm (3 in) exterior cover and a 51 mm (2 in) spacing between each row and column of strands. The maximum allowable tendon stress at transfer is taken as 0.74 fpu, and prestress losses are assumed to be 15% of the initial stresses after transfer.
5.2.2 Deflections

The short-term deflections are calculated using a bilinear moment-deflection method (Appendix A.9.5). This is necessary since the allowable flexural tensile stress under service loads is taken as $12\sqrt{f_c}$, which is considerably higher than the rupture strength of the concrete. The bilinear moment-deflection method consists of determining what fraction of the live load, when acting with the full dead load, results in a bottom tensile stress which exceeds the rupture strength of the concrete. A component of the total deflection is then calculated using this portion of the live load applied to the cracked section. The other components of the total deflection are calculated by applying the remaining portion of the live load, along with the applied dead loads, to the uncracked section. The long-term deflections are estimated using the multiplier method presented in Section 4.6.5 of the PCI Design Handbook [PCI 1992]. This method consists of applying prescribed factors to each component of the total deflection resulting from the different applied loads to account for time-dependent effects (Appendix A.9.5).

5.2.3 Flexural Design

The flexural design is based on allowable stresses under service loads and ultimate strength under factored loads. For the allowable stress portion of the design, the member stresses in the outer most fibers are checked against the allowable stresses defined by the ACI Code at a number of critical locations for the different stages of loading (Appendix A.8.2). Under composite action, the stresses in the slab are checked, in addition to those of the outer most fibers of the precast member. For the strength portion of the design, the strength is
checked at the critical stages of loading based on a simply supported member with the maximum moment occurring at mid-span (Appendix A.8.1). The different stages of loading and the critical stress locations are described below:

- The first stage of loading is taken as the time at which the prestressing force is released, while the member is still in the casting form, and is acted on only by its own weight. At this stage, the stresses are checked at the member ends, the mid-span, and the beginning of the first gap from the end of the member where the first abrupt change in cross-section occurs. The end stresses are taken to be critical at 50 strand diameters in from the end of the member, at the theoretical point of full prestress transfer [PCI 1992]. The tensile stresses encountered at this stage do not control the design, since supplementary top reinforcement is provided, where necessary, to accommodate the stresses which exceed the allowable stress.

- The intermediate stage of loading, which only applies to unshored construction, occurs when the member is supporting the load of its own weight, the weight of the members framing into it, and the weight of the topping slab without the benefit of composite action. At this stage the stresses are checked at mid-span, and at the first gap from the end of the member, with the assumption that the prestress force has been reduced by all potential losses to its effective value. The ultimate flexural strength for the given loading is also checked at this stage of loading.

- The final stage of loading, which applies to both shored and unshored
construction, occurs when the member is supporting all service loads with the aid of composite action. At this stage the stresses are checked at mid-span and the first gap from the end of the member. A check of the ultimate strength is also made at this stage.

The discontinuous cross-section resulting from the gaps provided in the upper portion of the stub girder, and the reliance on composite action, create certain unique design considerations. The flexural properties of the stub girder vary along the length of the member due to the changes in cross-section and also vary during construction due to the composite action provided by the cast-in-place slab. This problem was addressed by making certain estimates of the flexural properties as follows. For the non-composite stages, the flexural properties are taken as those of the continuous lower flange of the member, ignoring the contribution of the stubs. For the composite stages, the flexural properties are taken as those of the continuous lower flange combined with the contribution of the effective composite slab. Again, the contribution of the stubs is neglected. The one exception to these approximations occurs for the calculation of the critical end stresses under the initial stages of loading at transfer. For these calculations the full section properties are used which includes the contribution of the stubs. Excluding the contribution of the stubs in the end regions results in unrealistically high computed stresses which tend to control the design, leading to excessively large dimensions.
5.2.4 Shear and Torsion Design

Torsion considerations apply to unshored construction during the construction phase when only one side of the girder is loaded. For interior girders, this stage of loading is only temporary and consists of the weight of the double tees and the construction live load. The location of the first gap from the end of the member is taken as the critical region for torsion, since this is the smallest cross-section of the girder which experiences the highest combination of shear and torsion. In all cases investigated thus far however, the torsional strength of the concrete was adequate to resist the applied torsional loads without the need for additional torsion reinforcement (Appendix A.13).

Two types of critical shear forces are designed for, namely the vertical shear along the member and the horizontal shear at the interface between the cast-in-place slab and the tops of the stubs.

The vertical shear design is carried out by treating each stub and gap region individually, the stub regions being the full depth of the girder at those locations where a stub is located. When calculating the required reinforcement for the gap regions, it is assumed that the lower flange carries all the shear, ignoring the shear strength of the cast-in-place slab. The required stirrup spacing for the gap regions is set by the shear forces encountered at the first gap from the end of the member, since the shear forces for this gap are greater than for any of the other gap regions (Appendix A.12). The spacing required for this shear force is then used as the controlling spacing for all the gap regions, and is kept uniform throughout each region. It is understood, however, that less reinforcement would be required for the gap regions further from the end of the member, but, since System 1 has only three gap regions,
two of which carry the same shear force, it seemed reasonable to use the same shear reinforcement in all three regions. The required stirrup spacing for vertical shear in the stub regions is carried out in a similar manner, with the required spacing being controlled by the shear forces acting on the first stub at the end of the member (Appendix A.11).

The horizontal shear reinforcement required to provide composite action between the cast-in-place top flange and the girder is provided by extending the vertical shear reinforcement above the tops of the stubs into the cast-in-place slab. As a result, the required stirrup spacing in the stub regions is controlled by the minimum spacing required for either vertical shear or horizontal shear. The horizontal force to be accommodated at the interface is taken as the change in the compressive force in the cast-in-place flange across a given segment of the member. The magnitude of the compressive force in the flange at any section is determined by dividing the moment acting at the given section by the available moment arm. Since the change in compressive force over a given segment is directly related to the change of moment across the segment, the critical horizontal shear transfer region occurs over the first stub from the end of the member, where the rate of change in moment along the member (i.e. the shear force) is the greatest. The horizontal force to be taken by the first stub is calculated using the change in factored moment which occurs in the region from the end of the member, out to a distance equal to the length of the first stub plus the adjacent gap. The horizontal force is calculated at this point rather than at end of the first stub since, in the gap region, the cast-in-place flange is unable to transfer any horizontal forces to the girder. As a result, the stub must be able to accommodate the horizontal forces which develop across the length of the adjacent gap, in addition to those horizontal forces which develop directly
over the stub (Appendix A.11.4 (a)). Figure 5.1 illustrates how this horizontal force used for design is calculated for a simply-supported member.

The required stirrup spacing for horizontal shear is calculated using the horizontal shear strength which is described in Section 17.5 of the ACI Code (Appendix A.11.4 (b)). However, the ACI Code limits the horizontal force that can be carried according to the shear strength method. If this limit is exceeded, then the ACI Code requires that the shear friction method be used to determine the required reinforcement spacing (Appendix A.11.4 (c)). For either method, the required reinforcement to accommodate the horizontal force at the first stub is assumed to be uniformly spaced along the stub, and then this spacing is applied to all other stubs.

5.2.5 Design Differences for Shored and Unshored Construction

There are a few differences between the design of the stub girder for shored and unshored construction. One significant difference is the loadings which must be accommodated in the composite and non-composite stages. For unshored construction, the stub girder must be designed to meet the allowable stress and strength requirements for three distinct stages of loading, the initial non-composite stage at prestress transfer, the intermediate non-composite stage during construction, and the final composite stage, under the full service load. These stages of loading were described earlier in Section 5.2.3. In the non-composite state, only the continuous lower flange is available to carry load, and this provides less than half of the total flexural capacity of the composite girder. However, during this state, when the girder has less than half of its total flexural capacity, over 60% of the full
service loads are applied to the girder. As a result, the dimensions of the lower flange tend to be controlled by this intermediate non-composite stage of loading.

For shored construction, the stub girder must only be designed to accommodate two stages of loading, the initial non-composite stage at prestress transfer, and the final composite stage under the full service load. As a result, the shored stub girder is not required to carry any loads, other than its own weight, until full composite action has developed. This allows for a more efficient final design, since the girder proportions are tailored for the final in-service stage. It was found that the required depth of the shored stub girder tends to be about 20% to 25% less than the required depth of the unshored stub girder.

There are also differences in the torsion design between shored and unshored construction. For an interior girder, a torsional load occurs during construction when only one side of the girder has been loaded. This situation is not a concern with shored construction since the member is assumed to be supported against twisting by the shoring.

Another difference between the design for shored and unshored construction is the horizontal loads which must be transferred between the cast-in-place slab and the precast stubs. As was described in Section 5.2.4, the horizontal force to be designed for is calculated by dividing the change in moment across a given critical section by the available moment arm. For the shored stub girder however, the depth of the girder tends to be appreciably smaller than that of the unshored stub girder (20% to 25% smaller), as a result, the available moment arm is also appreciable smaller. The smaller moment arm leads to higher horizontal forces at the interface of the cast-in-place slab and the stubs. These higher horizontal forces for the shored stub girder means that the required stirrup spacing for the stub regions is likely to be
controlled by horizontal shear transfer which is not usually the case with the unshored stub girder.

5.3 STUB GIRDER DESIGN SUMMARY

This section presents the results of the stub girder design based on the approach described in Section 5.2. A design was made for both shored and unshored construction, and the results of both are summarized here. Each summary provides information on the dimensions of the stub girder and both the prestressed and non-prestressed reinforcement specifications.

The bay dimensions used for the calculations were 9.75 m (32 ft) in the direction of the girders and 12.19 m (40 ft) in the direction of the double tees. The 9.75 (32 ft) column spacing in the direction of the girders was chosen because it conveniently accommodates four 2.44 m (8 ft) double tees. The required column size was calculated to be 457 mm x 457 mm (18 in x 18 in), based on an interior bay of a 4 story structure with full height columns. The calculations carried out for the unshored version are presented in Appendix A. Similar calculations were carried out for the shored version as well, but examples of these calculations are not provided.

5.3.1 Stub Girder Design Summary for Unshored Construction

Figure 5.2 shows an elevation of the unshored stub girder, and Figure 5.3 shows several cross-sections of the girder.
5.3.1.1 Dimensions

The calculations carried out for the given design parameters resulted in a stub girder that has an overall, non-composite depth of (hc) of 965 mm (38 in). This depth is a function of a lower flange depth (h2c) of 508 mm (20 in) and a stub depth (h1c) of 457 mm (18 in). The bearing surfaces (b2c) for the double tees which rest on the lower flange are 152 mm (6 in) wide, and the width of the stubs (b1c) are 559 mm (22 in) which results in an overall girder width (bc) of 864 mm (34 in). Each stub is 1676 mm (66 in) long which provides three 864 mm (34 in) gaps in the girder which is 9.30 m (30.5 ft) long from column face to column face.

In some cases, the dimensions presented for the stub girder are not the minimum dimensions which could have been used without violating any of the design criteria. For dimensions such as the height of the stubs, a 406 mm (16 in) stub height could have been used, but, to avoid dapping of the double tees, which are 508 mm (20 in) deep, a stub height of 457 mm (18 in) was used instead (see Figure 4.4.). It was also found possible to reduce the required depth of the continuous lower flange by increasing the overall width of the girder. This adjustment increases the number of strands which can be placed in the member at any given eccentricity, and slightly increases the moment of inertia. However, this increase in girder width significantly increases the total weight of the girder. One potentially key advantage of the deeper narrower version is that it may be more compatible with current inverted tee forms. For the stub girder presented in this section, the deeper, narrower version was chosen.
5.3.1.2 Prestressed Reinforcement

The cross-sections shown in Figure 5.3 illustrate the strand pattern used throughout the length of the member. A total of nineteen, 13 mm (1/2 in), seven-wire, low relaxation strands were used at an initial stress of 1330 MPa (193 ksi), which results in an initial prestress force of 2502 kN (563 kips). A straight strand pattern was used without any bond released tendons. Fifteen strands were placed in the furthest available row from the centroidal axis, and four additional strands were placed in the second furthest row from the centroidal axis. This strand pattern and tendon stress is the minimum number of strands and maximum possible stress that could be used with the given member geometry which would not violate any of the allowable concrete stresses, but would still provide the member with adequate strength at ultimate loads. This is based on the design calculations presented in Appendix A.

5.3.1.3 Non-Prestressed Reinforcement

The required stirrup spacing in the stub regions is 610 mm (24 in) using #5, grade 50 stirrups. This spacing is based on the maximum shear forces encountered at the first stub from the end of the member and is controlled, in this case, by the maximum spacing allowed by the ACI Code (Appendix A.11). However, taking into account the actual length of the stub, a spacing of 546 mm (21.5 in) is suggested. This will provide a concrete cover of 20 mm (3/4 in) at either end of the stub, which is greater than the 16 mm (5/8 in) minimum required by the provisions of Section 7.7.2 of the ACI Code.

The required stirrup spacing in the gap regions, according to the design calculations, is 381 mm (15 in), using #5, grade 50 stirrups, similar to those used for the stub regions
However, to better accommodate the length of the gap region, a spacing of 280 mm (11 in) is suggested for fabrication.

The required supplementary longitudinal top reinforcement needed to carry the tensile forces resulting from the initial stage of loading at prestress transfer is calculated at three critical sections. The first is the top of the first stub at the end of the member, the second is the top of the first gap from the end of the member, and the third is the top of the center gap. Reinforcement is only required if the concrete tensile stresses are higher than that allowed by the ACI Code, Section 18.4 (Appendix A.14.5). According to the design calculations, the first stub requires six #5 bars, the first gap requires three #5 bars, and the center gap requires two #5 bars. For adequate development of this reinforcement, it is recommended that a 90° hook be provided at the end of the bars with a 203 mm (8 in) hook development, as specified by Section 12.5 of the ACI Code. The cross-sections illustrated in Figure 5.3 show the different longitudinal reinforcing patterns used throughout the girder.

5.3.2 Stub Girder Design Summary for Shored Construction

Figure 5.4 shows an elevation of the shored stub girder. Figure 5.5 shows the different cross-sections of the girder.

5.3.2.1 Dimensions

The calculations carried out for the given design parameters resulted in a stub girder that has an overall, non-composite depth (hc) of 762 mm (30 in). This depth is a function of a lower flange depth (h2c) of 305 mm (12 in) and a stub depth (h1c) of 457 mm (18 in). The bearing surfaces (b2c) for the double tees, which rest on the lower flange, are 152 mm (6 in)
wide, and the width of the stubs (b1c) are 457 mm (18 in), resulting in an overall girder width (bc) of 762 mm (30 in). Each stub is 1676 mm (66 in) long, providing three 864 mm (34 in) gaps in the girder, which is 9.30 m (30.5 ft) long from column face to column face.

As was the case with the dimensions for the unshored stub girder, the dimensions determined for the shored stub girder are not, in all cases, the minimum dimensions which could have been used without violating any of the design criteria. For example, a shallower stub could be used if dapping of the double tees is provided. Certain dimensions could also be decreased if certain other dimensions were increased, depending on the needs of the particular design. For example, the depth of the lower flange could be reduced if the width of the either the stubs or the double tee bearing surfaces are increased. However, the dimensions presented in this section represent the most efficient design, in terms of weight, and fabrication efficiency, as well as providing a cross-section which has the most potential for being compatible with current inverted tee forms.

5.3.2.2 Prestressed Reinforcement

The cross-sections shown in Figure 5.5 illustrate the strand pattern used throughout the length of the member. A total of fifteen, 13 mm (1/2 in), seven-wire, low relaxation strands were used at an initial stress of 1282 MPa (186 ksi), which results in an initial prestress force of 1900 kN (427 kips). A straight strand pattern was used, without any bond released tendons. Thirteen strands were placed in the furthest available row from the centroidal axis, and two additional strands were place in the second furthest row from the centroidal axis. As was the case with the unshored stub girder, this strand pattern, and tendon stress, is the minimum number of strands, and maximum possible stress, that could be used
with the given member geometry which would not violate any of the allowable concrete stresses, but would still provide the member with adequate strength at ultimate loads.

5.3.2.3 Non-Prestressed Reinforcement

The required stirrup spacing in the stub regions is 254 mm (10 in), using #5, grade 50 stirrups, which is based on the maximum shear forces encountered at the first stub at the end of the member. However, for fabrication, a spacing of 203 mm (8 in) is suggested, since this will better accommodate the given stub length while accounting for the minimum cover requirements specified by the ACI Code, Section 7.7.2.

The required stirrup spacing in the gap regions, according to the design calculations, is 145 mm (5.7 in), using #5, grade 50 stirrups. For fabrication, however, a spacing of 140 mm (5.5 in) is suggested in consideration of the given gap lengths.

The required supplementary longitudinal top reinforcement, needed to carry the tensile stresses resulting at the initial stage of loading at the time of prestress transfer, is calculated at three critical sections. The first is the top of the first stub at the end of the member, the second is the top of the first gap from the end of the member, and the third is the top of the center gap. Reinforcement is only required if the concrete tensile stress is higher than that allowed by the ACI Code, Section 18.4. According to the design calculations, the first stub requires three #5 bars, and the other two critical locations do not require any supplementary top reinforcement. For adequate development of this reinforcement, it is recommended that a 90° hook be provided at the end of the bars with a 203 mm (8 in) hook development, as specified by Section 12.5 of the ACI Code. The cross-sections illustrated in Figure 5.5 show the different longitudinal reinforcing patterns used throughout the girder.
Figure 5.1. Free-body diagram of the first stub at the end of the girder illustrating the horizontal shear force acting at the stub and slab interface.
Figure 5.2. Elevation of the unshored version of the stub girder designed for System 1.
Figure 5.3. Cross-sections for the unshored stub girder designed for System 1: a) cross-section of first stub at the end of the girder (Section A-A); b) cross-section of the first gap from the end of the girder (Section B-B).
Figure 5.3 (Cont.). c) cross-section of the second stub from the end of the girder (Section C-C); d) cross-section of the gap at the mid-span of the girder (Section D-D).
Figure 5.4. Elevation of the shored version of the stub girder designed for System 1.
Figure 5.5. Cross-sections for the shored stub girder designed for System 1: a) cross-section of first stub at the end of the girder (Section A-A); b) cross-section of the first gap from the end of the girder (Section B-B).
Figure 5.5 (Cont.). c) cross-section of second stub from the end of the girder (Section C-C); d) cross-section of the gap at the mid-span of the girder (Section D-D).
Chapter 6

Alternate Configurations for System 1

6.1 INTRODUCTION

This chapter discusses some of the alternate configurations considered for System 1. These alternate configurations include different bay dimensions for both shored and unshored construction and the use of moment resistant connections.

6.2 STUB GIRDER DESIGNS FOR DIFFERENT BAY DIMENSIONS

A variety of different bay configurations, other than those used for the design presented in Chapter 5, can be accommodated with System 1. This section presents a discussion of the different bay dimensions which have been examined using 2.44 m (8 ft) wide double tees. A summary of these different bay dimensions is provided in Table 6.1 which shows the required depth of the double tees, and some of the characteristics of the stub girder required for the given bay dimensions. These characteristics include the dimensions of the stub girder, the number of strands used, the effective prestress force, the average weight per unit length, and the estimated immediate and long-term deflections. The cross-section dimensions are given in Table 6.1 in terms of the variables that are defined in Figure 6.1.

Six different pairs of bay dimensions were examined, consisting of combinations of two different girder lengths and four different double tee lengths. The two girder lengths of 6.86 m (22.5 ft) and 9.30 m (30.5 ft) were based on bay dimensions which could
accommodate three and four double tees side-by-side, assuming the use of 457 mm x 457 mm (18 in x 18 in) multistory columns. The four double tee spans were chosen with intent of providing an idea of the practical range of floor member spans that could be used with System 1. The required use of dapped double tees is not recommended by the industry advisors, since it decreases the cost-effectiveness of the system. For several of the bay dimension combinations, however, dapping was provided to better illustrate the capabilities of the stub girder.

The analysis and design calculations used to determine the values presented in Table 6.1 are based on the stub and gap lengths shown in the table. These lengths were defined such that all the stubs would be the same, and all the gaps would be the same, and the gaps would fall between the stems of the double tees. This uniformity is not a requirement, but the design calculations, presented in Appendix A, do assume constant lengths for the stubs and gaps. The arrangement of stubs and gaps is really a function of the particular application and the discretion of the designer. For example, if wider double tees are used, it may be most effective to vary the length of the stubs or gaps along the member in order to provide a design that best meets the needs of the designer, and still provide a stub and gap pattern that is symmetrical about the center line of the girder.

The use of double tees that are wider than the 2.44 m (8 ft) width specified in Chapter 4 was one of the recommendations of several of the industry advisors. They recommend using of 3.05 m (10 ft) or 3.65 m (12 ft) double tees, since these widths, in their experience, are becoming more common than the 2.44 m (8 ft) width. The designs carried out to date, however, have all been based on 8 ft wide double tees. For designs using wider widths, the
design and analysis will be essentially the same as it is for the 2.44 m (8 ft) widths. The stem loads associated with the wider double tees, however, will be higher than those of the 2.44 m (8 ft) double tees, due to the larger tributary width. The local effects of these stem loads may have to be looked at in more detail, but it is unlikely that these higher loads will present a problem, since they are presently accommodated by inverted tee girders.

6.3 MOMENT RESISTANT CONNECTIONS FOR THE STUB GIRDER

Another possible variation for System 1 is the use of moment resistant connections at the ends of the stub girders. The negative moment developed in the connections would provide the benefit of reducing the maximum mid-span moment for which the stub girder would have to be designed.

The Precast Concrete Institute (PCI) provides suggested details for a variety of different types of moment resistant connections, PCI [1988]. One of these connection details, namely the GC21 connection, provides moment resistance at the girder and column connection by means of post-tensioning the end of the girder on one side of the column to the end of the girder on the other side. The stub girder appears to offer some unique opportunities for this particular connection. If a duct is provided through the end stub of each girder (and through the column for situations where continuous column construction is used), then the gap adjacent to the end stub can provide easy access to the end of the duct, for the placement of the post-tensioning strand, and the application of the jacking force. Figure 6.2 shows an elevation of a possible moment connection for the stub girder using this method. If temporary supports for the ends of the girder are used, or if shoring is provided, to support
the girder prior to post tensioning, it may be possible to eliminate the use of corbels, as shown in the Figure 6.2. Another possible method of providing a moment resistant connection for the stub girder, which is similar to the PCI connection GC8, is to make the stub girder continuous over the column. This method, however, can only be used with single-story column construction, and it requires that ducts be provided in the girder for the purpose of splicing the rebar between columns. A third possible method is to provide negative moment steel in the cast-in-place slab and weld the bottom of the girder to the column corbel. This type of connection is similar to the PCI connection GC20, and could be used with either single-story or multi-story column construction.

The connections described above are but a few of the possibilities for moment resistant connections for the stub girder. It is likely that other suitable methods exist, but more work is needed in this area.
<table>
<thead>
<tr>
<th>Bay Dimensions (ft)</th>
<th>Girder Length L (ft)</th>
<th>Double Tee Depth (in)</th>
<th># of Gaps</th>
<th>Gap Length (in)</th>
<th>Stub Length (in)</th>
<th>Stub Girder Cross-Section Dimensions (in)</th>
<th>Avg. Girder Weight (plf)</th>
<th># of Strands</th>
<th>Prestress Force Pe (kips)</th>
<th>Deflections (in)</th>
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<td>30.5</td>
<td>16</td>
<td>3</td>
<td>34</td>
<td>66.0</td>
<td>h1c: 14, h2c: 18, b1c: 20, b2c: 6</td>
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<td>18</td>
<td>426</td>
<td>-0.59 -0.99</td>
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<tr>
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<td>16</td>
<td>3</td>
<td>34</td>
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<tr>
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<td>19</td>
<td>546</td>
<td>-0.47 -0.86</td>
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<td>3</td>
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<td>15</td>
<td>363</td>
<td>0.48 0.31</td>
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<tr>
<td>32 x 46 u</td>
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<td>3</td>
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<tr>
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<td>22.5</td>
<td>24*</td>
<td>2</td>
<td>34</td>
<td>67.3</td>
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<td>13</td>
<td>318</td>
<td>0.50 0.49</td>
</tr>
<tr>
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<td>22.5</td>
<td>24*</td>
<td>2</td>
<td>34</td>
<td>67.3</td>
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<td>18</td>
<td>428</td>
<td>-0.31 -0.56</td>
</tr>
<tr>
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<td>24*</td>
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<td>34</td>
<td>67.3</td>
<td>h1c: 18, h2c: 10, b1c: 18, b2c: 6</td>
<td>565</td>
<td>13</td>
<td>318</td>
<td>0.49 0.47</td>
</tr>
</tbody>
</table>

All designs based on a superimposed dead and live load of 15 psf and 50 psf respectively.

* = Double tees must be dapped for the given stub girder
u = Unshored construction
s = Shored construction
A = Immediate deflections
B = Long-term deflections

1 foot = 0.3048 meters
1 inch = 25.4 millimeters
1 kip = 4.448 kN
1ksi = 6.895 MPa
1 lb/ft = 14.59 N/m

Table 6.1. Summary of stub girder dimensions, prestress force, and deflections for different bay sizes using 2.44 m (8 ft) wide double tees.
Figure 6.1. Stub girder cross-section.

Figure 6.2. Post-tensioned moment connection for the stub girder.
7.1 INTRODUCTION

The research presented in this report is part of a project titled Development of New Floor Framing Systems for Gravity Loads. The project has two broad objectives:

1. To develop new structural floor systems for gravity loads; and,
2. To develop a methodology for the systematic comparison and evaluation of different structural floor systems.

The present work focuses on the development of new precast concrete floor systems for gravity loads for application in office buildings with regular spacing of columns or bearing walls.

The project objectives have been separated into three tasks as follows:

Task 1: Assess current and emerging precast concrete structural floor systems

Task 1.1 - survey of current and emerging precast structural systems;
Task 1.2 - develop assessment criteria to evaluate precast structural systems;
Task 1.3 - apply assessment criteria to precast structural systems in the survey.
Task 2: Develop new concepts for precast structural floor systems

Task 2.1 - identify opportunities for new precast structural systems;
Task 2.2 - propose new concepts for precast structural systems;
Task 2.3 - apply assessment criteria to new precast structural systems.

Task 3: Detailed development of most promising new concepts from Task 2

The results of the work performed up to and including Tasks 2.1 were presented in an ATLSS report titled "Identification and Preliminary Assessment of Existing Precast Concrete Floor Framing Systems" [Prior, Pessiki, Sause, Slaughter, and van Zyverden 1993]. This report presents the results of Task 2.2 - propose new concepts for precast structural systems, and Task 3 - detailed development of most promising new concepts from Task 2. This report, however, does not complete Task 3, nor does this report address the application of the assessment criteria (Task 2.3) developed by Prior et al. [1993].

7.2 SUMMARY

The work presented in this report produced several important advances in the development of new precast concrete floor systems. The development process described in Chapter 3 produced five concepts for new or improved precast concrete floor framing systems and a new precast stub girder. These five concepts are described in Chapter 4 and summarized below in Section 7.2.2. From these five systems, System 1 was determined to be the most promising by a panel of industry advisors. A design methodology was developed for stub girder which is the key component of System 1. A design of the stub girder for both
shored and unshored construction is summarized below in Section 7.2.3. Several alternative configurations for System 1 were also considered, the results of which are summarized below in Section 7.2.4. Although a more thorough investigation of the stub girder is still needed, a design aid was developed for the stub girder which makes use of a computer spreadsheet specifically set up for the design of the stub girder used in System 1. A discussion of this stub girder design aid is presented below in Section 7.2.5.

7.2.1 Classification of Floor Framing Systems

By re-examining the existing precast concrete floor systems identified by Prior et al. [1993] and the conclusions made about opportunities for developing new precast structural systems, it was found that most floor systems could be generally categorized by two particular characteristics. The first characteristic is the manner in which the structural system accommodates the service systems. The second characteristic that can be used to classify floor systems is the relative amounts of field work and prefabrication work required by a system.

As discussed in Chapter 3, this leads to the following four general categories of floor systems:

1. Layered, field oriented systems
2. Layered, prefabrication oriented systems
3. Integrated, field oriented systems
4. Integrated, prefabrication oriented systems
While any particular system (existing or proposed) may not fall clearly into one particular category, the above categorization does provide a convenient frame of reference to view existing and proposed systems.

The four classes of floor systems described above each offer certain important advantages and disadvantages. As a result, the comprehensive approach followed for the development of new or improved floor system concepts attempted to cover these four classes of floor systems, in addition to addressing the opportunities defined by Prior et al. [1993].

7.2.2 Concepts for Structural Floor Framing Systems

SYSTEM 1 - Double tees with stub girders

System 1 consists of precast stub girders and precast double tee floor members. This system is similar to the conventional double tee floor system except that the inverted tee girder is replaced with a precast stub girder which is similar to the steel stub girder [Colaco 1972]. This system is classified as an Integrated, prefabrication oriented system according to the system classifications described in Chapter 3. By means of the double tee floor members, System 1 offers the capability for long spans, and provides a significant amount of floor area with the placement of each member. The openings in the stub girder allow for the horizontal passage of services within the structural depth of the floor system which helps to minimize the total floor depth.

SYSTEM 2 - Cast-in-place slab on channel members with inverted tee girders

System 2 is composed of inverted tee girders supporting 1.52 m (5 ft) wide
channel members spaced at a maximum of about 4.57 m (15 ft) on centers. The channel members in turn support corrugated steel decking that acts as a stay-in-place form for a cast-in-place slab. This system is classified as a *Layered, prefabrication oriented system* according to the classifications described in Chapter 3. With the ability to vary the span of the cast-in-place slab between channel members, System 2 offers the advantage of better accommodating irregular or varying building geometries. The space between channels can provide a convenient location for openings which enable the vertical passage of services thorough the floor system. This space can also be used for the placement of some of the larger components of the HVAC system, such as diffusers and variable-air-volume (VAV) boxes.

**SYSTEM 3 - Cast-in-place slab on channel members with precast stub girders**

System 3 utilizes the same channel and slab construction as System 2, but replaces the inverted tee girder with a stub girder similar to the one used in System 1. This system is classified as an *Integrated, prefabrication oriented system* according to the classifications described in Chapter 3. System 3 offers all the same advantages of System 2, but with the additional advantages provided by the use of the stub girder which can help to minimize the total floor depth.

**SYSTEM 4 - Prestressed joists with a cast-in-place slab and inverted tee girders**

System 4 is an adaptation of the Prestressed Joist systems by Cast-Crete of Tampa, and Prestressed Systems Industries of Miami (PSI). The system consists of inverted tees supporting prestressed rectangular joists which in turn support precast
planks. The precast planks serve as forms for a cast-in-place composite slab. This system is classified as a *Layered, field oriented system* according to the classifications described in Chapter 3. With the ability to vary the joist spacing, System 4 offers the ability to achieve relatively long spans with minimum floor depth. This variability of joist spacing also provides flexibility for accommodating irregular or varying building geometries. Requiring minimum prefabrication and being more field-labor oriented, this system also has the potential of providing certain cost-saving advantages in areas where low-cost field labor is available.

**SYSTEM 5 - Prestressed joists with cast-in-place slab and precast stub girders**

System 5 consists of precast stub girders supporting prestressed joists which in turn support precast planks. The precast planks serve as forms for a cast-in-place composite slab. This system is classified as an *Integrated, field oriented system* according to the classifications described in Chapter 3. System 5 offers all the same advantages of System 4, but with the additional advantage of minimal total floor depth provided by the use of the stub girder.

### 7.2.3 Shored and Unshored Stub Girder Designs for System 1

The design calculations for both the shored and unshored versions of the stub girder were based on 2.44 m (8 ft) wide double tees and an industry recommended bay size of 9.75 m x 12.19 m (32 ft x 40 ft) which requires the use of 457 mm (18 in) deep double tees. Examples of the design calculations carried out for the unshored stub girder are presented in
Appendix A. Similar calculations were carried out for the shored version, but examples of these calculations are not provided. Both versions of the stub girder provide three openings for services which are each 864 mm (34 in) long and 406 mm (16 in) deep. These openings are intended to provide adequate space for the passage of all services which include nothing larger than the secondary portions of the HVAC system. The total depth measured from the top of the slab to the bottom of the girder for the unshored version is 1067 mm (42 in), and for the shored version it is 762 mm (30 in). A more detailed description of cross-section dimensions and the required prestressed and non-prestressed reinforcement is provided in Chapter 5.

7.2.4 Alternate Configurations for System 1

Several alternate configurations were considered for System 1. These alternate configurations included different bay dimensions for both shored and unshored construction and the use of moment resistant connections.

The bay dimensions examined consisted of six different combinations of girder and floor member lengths using 2.44 m (8 ft) wide double tees. Wider double tees could also have been used, as discussed in Chapter 6, but the examinations carried out for this report have been limited to 2.44 m (8 ft) double tees. A summary of the different bay dimensions is provided in Chapter 6 which show the required depth of the double tees, and some of the characteristics of the stub girder required for the given bay dimensions. These characteristics include the dimensions of the stub girder, the number of strands used, the effective prestress force, the average weight per unit length, and the estimated immediate and long-term
deflections. This examination of bay dimensions provided an indication of the capabilities of
the stub girder, and the differences in girder depth required for shored and unshored
construction.

The use of moment resistant connections at the ends of the stub girders has also been
considered. The negative moment developed in the connections would provide the benefit
of reducing the maximum mid-span moment for which the stub girder would have to be
designed. Although the use of moment resistant connections has not been looked at in detail,
Chapter 6 indicates three different standard PCI connection details that could be readily
adapted for use with the stub girder.

7.2.5 Stub Girder Design Aid

A computer spreadsheet design aid was developed to carry out the numerous design
calculations necessary for the shear and flexural design of the stub girder based on the
provisions of the ACI Code. The spreadsheet is easily modified, and enables all the design
calculations to be instantaneously recalculated each time a user-defined variable is changed.

Design, using the spreadsheet, is an iterative process which requires user input to
define the design parameters. The required input includes information such as material
strengths, type and size of both the prestressed and non-prestressed reinforcement, the design
loads, the member geometry, the bay dimensions and the arrangement of the prestressed
reinforcement. Using this information, the spreadsheet calculates the stresses at the critical
points, the flexural strength, the estimated deflections, and the required shear and torsion
reinforcement.
Appendix A presents a detailed description of the procedure to be followed using the design aid, as well as a design example using the design parameters for the unshored stub girder design presented in Chapter 5.

7.3 FUTURE WORK

The detailed development of the most promising new concept from Task 2, namely System 1, still requires more work in a number of areas pertaining to the stub girder. The calculations carried out for the design of the stub girder, as presented in Appendix A, make use of the ACI Code to address the basic requirements for shear and flexure. This method, however, does not account for the unique detailing issues associated with the presence of the discontinuities created by the gaps. Future research needs are listed below:

1. The detailing of the steel reinforcement in the cast-in-place slab placed over the stub girder needs to be investigated. Both the strength and stability of the slab, particularly in the gap regions, needs to be investigated. Previous research on steel stub girders with cast-in-place slabs showed that failure occurred by crushing of the slab at the interior of the first stub, followed by shear splitting of the slab along the length of the stub [Kullman and Hosain 1985]. Similar failure modes were observed in a full-scale test of a steel stub girder by Buckner, Deville and McKee [1981] and in small-scale tests by Nadasky and Buckner [1985].

2. Chapter 5 and Appendix A described the design of reinforcement in the stub and gap regions of the girder, considering the requirements for vertical and horizontal shear. A more
detailed analysis of the requirements for reinforcement still must be performed to ensure
adequate transfer of forces across the stub and gap regions.

3. Additional research should be performed to determine the practical limitations on stub
height and gap lengths for the stub girder cross-sections, including cross-sections that are
compatible with currently used inverted tee forms.

4. Chapter 6 described the possibility of using moment connections with the stub girder. This
can be achieved in a variety of ways, and this possibility needs to be explored in greater detail.

5. Additional consideration should be given to practical field issues such as efficient methods
for forming the cast-in-place slab over the stub girder.

6. The behavior of the stub girder concept needs to be verified through a program that
includes experimental testing.
REFERENCES

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Appendix A

Stub Girder Design Aid

A.1 INTRODUCTION

This Appendix presents a detailed description of the procedure for designing an unshored stub girder for System 1 using a spreadsheet design aid written specifically for this task. The unshored stub girder presented in Chapter 5 is used as the example to explain this spreadsheet.

To effectively explore the basic limitations of the stub girder and to develop efficient, feasible designs, it was necessary to devise a means of handling the numerous design calculations that must be carried out each time one or more of the design parameters is changed. The spreadsheet design aid was created for the purpose of providing such a means. It is an easily modified design tool which enables a designer to quickly arrive at a reasonable design for the stub girder for almost any application appropriate for the stub girder. The version of the design aid presented in this appendix has been specifically tailored to the design of the stub girder for System 1 which uses double tee floor members, but the design aid could be modified for use with other types of floor members.

The design aid makes use of the spreadsheet function which enables a variable to be defined such that it will take on any value entered into the adjacent cell. This function makes it possible to write all equations in variable form. The design aid also makes frequent use of the conditional IF THEN function which provides a powerful means of handling the many
conditional calculations that must be made when designing by code equations. All cells which require a user-specified value are indicated by the use of shading. All other cells, which are not shaded, present values that are automatically calculated within the spreadsheet each time any user-specified variable is changed.

A.2 OVERVIEW

The stub girder design aid is divided up into the twelve sections listed below. Each section handles a different aspect of the design process, as indicated by the title of the section.

1. DESIGN LOADS (Section A.4)
2. GEOMETRIC MEMBER DEFINITION (Section A.5)
3. FLEXURAL DESIGN SPECIFICATIONS (Section A.6)
4. MAGNEL DIAGRAM (Section A.7)
5. FLEXURAL STRENGTH, STRESS, AND DEFLECTION CHECKS (Section A.8)
6. FLEXURAL DESIGN AND ANALYSIS CALCULATIONS (Section A.9)
7. CALCULATION OF INITIAL PRESTRESS FORCE LIMITS (Section A.10)
8. SHEAR REINFORCEMENT DESIGN FOR THE STUB REGIONS (Section A.11)
9. SHEAR REINFORCEMENT DESIGN FOR THE GAP REGIONS (Section A.12)
10. TORSION REINFORCEMENT DESIGN (Section A.13)
11. STUB GIRDER DESIGN SUMMARY (Section A.14)
12. NOTATION (Section A.15)

Design, using the spreadsheet, is an iterative process which requires user input to define the design parameters. The required input includes information such as material strengths, type and size of both the prestressed and non-prestressed reinforcement, design loads, member geometry, bay dimensions and the arrangement of the prestressed reinforcement. Using this information, the spreadsheet carries out all calculations necessary for the design of the stub girder, such as the stresses at the critical points, the flexural strength, the estimated deflections, and the required shear and torsion reinforcement at all critical sections. A check of these values against code-specified allowable limits is also provided, indicating to the designer whether any violations of these limits exist.

Definition of the most efficient flexural design possible requires iteration to arrive at a suitable combination of prestress force and steel eccentricity which will provide the minimum possible member dimensions capable of satisfying all the design requirements. The dimensions and prestressing specifications are each adjusted until the member stresses at all critical locations do not exceed the allowable stresses, the deflections are tolerable, and the member has adequate strength to accommodate the ultimate loads. Because, the adjustment of any of the section dimensions will also change the steel eccentricity for a given strand pattern and possibly even the number of strands which can be placed in any given row of strands. As a result, the design process can become quite challenging when the most efficient cross section is sought.
To help simplify challenges associated with the design process, two key design aids have been incorporated into the spreadsheet. The first is the automatic calculation and application of the maximum allowable prestress force which can be used without violating any of the allowable concrete stresses for any given strand pattern and member geometry. The second key design aid is the calculation of a Magnel diagram. The Magnel diagram is a graphical illustration of the different combinations of prestress force and steel eccentricity which can be used without violating the allowable stresses at the critical stages of loading for a given set of member dimensions. With the help of these two design aids, arriving at an efficient design for a given stub girder application can usually be achieved after only a few iterations. The suggested design procedure, using the spreadsheet, is presented below.

A.3 DESIGN PROCEDURE

This section provides a description of the suggested design procedure to be followed using the spreadsheet stub girder design aid. The procedure described below is specifically tailored for the design of stub girder used in System 1. However, the procedure could be applied to any other appropriate use of the stub girder by adjusting the way the design loads are calculated to reflect the particular application. Section A.14 provides a summary of the design and Section A.15 provides definitions for all the variables used in the spreadsheet.

Step 1 - Definition of design loads

This step involves defining the value of eight different variables found in the \textit{DESIGN LOADS} section (Section A.4).
The first variable **XB** defines the column to column spacing in the direction of the girders. The second variable **YB** defines the column to column spacing in the direction of the floor members. The third variable **Floor** is for specifying the double tee to be used which is capable of accommodating the specified floor span. The fourth variable **Nfloor** defines the number of double tees framing into one side of the girder. The fifth variable **ts** defines the desired depth of the cast-in-place topping slab to be placed on the double tees. The sixth variable **Qo** defines the equivalent self weight of the chosen double tee in terms of pounds per square foot of floor area provided by the member. The seventh and eight variables **Qdce** and **QI** define the unfactored superimposed dead and live load respectively.

Using these values, the loads delivered to the stub girder, through the stems of the double tees, are calculated (Section A.4.2) and then translated into equivalent uniform loads applied to the stub girder (Section A.4.3).

**Step 2 - Definition of the stub girder dimensions**

This step involves defining the values of thirteen variables in the **GEOMETRIC MEMBER DEFINITION** section (Section A.5).

The first variable **Member** is for defining the title of the stub girder being designed. However, the title of the girder incorporates the key dimensions which define its cross-section, therefore this variable cannot really be defined until the dimensions of the girder have been chosen. Once the title has been entered it is automatically presented at the top of each of the eleven different sections of the design aid.

The next three variables pertain to the definition of the cast-in-place compression
The first variable $tc$ defines the compression flange depth between stubs, and is typically taken as 5.0 inches. This dimension, however, should only reflect the depth of the cast-in-place concrete, and should not include the depth of the forms used to support the slab during casting. The second variable $ta$ defines the depth of the slab above the precast stubs, and should not be less than 4.0 inches. The 4.0 inch minimum is based on studies of the anchorage of stirrups in a thin cast-in-place slabs [Mattock 1987]. The third variable $W_{cip}$ defines the length of the top flange which must be removed from each end of the double tees, as well as the length of the stems which must be coped a depth of about 2.0 inches. The purpose of removing the top flange of the double tees and coping the top 2.0 inches of the stems is to provide an adequately deep compression flange for the ultimate stages of loading of the girder. Figure 4.4 shows a cross-section of this cast-in-place compression flange.

The next five variables to be defined all pertain to the dimensions of the stub girder (Section A.5.2). The first variable $L$ defines the actual length of the stub girder, and is equal to the distance from column face to column face in the direction of the girders. The second variable $h_{1c}$ defines the height of the stubs which is primarily a function of the depth of the double tees used and the depth of the opening desired for the passage of services through the girder (see Figure 4.4). The third variable $h_{2c}$ defines the depth of the continuous lower flange of the stub girder. This depth is critical since, during the non-composite stages of loading, this portion of the member must carry the self weight of the girder plus the dead weight of all the members framing onto the girder combined with the weight of the wet topping slab. The fourth variable $b_{1c}$ defines the width of the stubs. The fifth variable $b_{2c}$
defines the width of the double tee bearing surface and, for that reason, a minimum of 6.0 inches is recommended.

The last three variables define the length and arrangement of the stubs and gaps (Section A.5.2). The first variable Gap defines the length of the gaps. The second variable Ngap defines the number of gaps, and the third variable Nstub defines the number of stubs. For the convenience of design and fabrication, the length of the stubs are all equal, and the length of the gaps are all equal. The actual length of the stubs and gaps is a function of the particular design.

With the dimensions defined, the spreadsheet will automatically calculate all the non-composite and composite section properties for both the stub and gap regions. These calculations are carried out in Sections A.5.3 and A.5.4.

The first time the dimensions are defined in the design process, many of the dimensions specified are best guess estimates. These estimates are refined through iteration until a member which satisfies all the design requirements is found.

Step 3 - Definition of the flexural design specifications

This step involves the definition of ten variables which are all found in the FLEXURAL DESIGN SPECIFICATIONS section (Section A.6).

The first variable xr defines a point along the girder measured from the end. This variable provides the designer with a means of specifying a critical stress location to be checked in addition to the end region and centerline of the girder (Section A.6.1 (b)).

This additional stress check can be quite important. The spreadsheet is set up to
automatically check only the stresses at the end of the girder for initial conditions and at mid­span for service conditions, since these locations being the critical stress regions for a simply supported member with straight strands and a continuous cross-section. However, the stub girder has an abrupt reduction in cross-section at the location of the gaps so there is the potential for other critical regions. The stress calculations carried out for point \( x_r \) do not include the contribution of the stubs, and therefore provide an effective means of checking the stresses in the region of the first gap. It is therefore recommended that \( x_r \) be used to check these stresses as a routine part of the design process.

If the design requires the use of bond released tendons, \( x_r \) offers a means of simultaneously checking several different critical load combinations that can result when bond released tendons are used. However, in order to make use of this capability, \( x_r \) must be set equal to the intended bond release length. This is necessary because the stress equations associated with \( x_r \) assume the full strand pattern to one side of \( x_r \) and the reduced strand pattern to the other side. However, the designer does not have to explicitly specify whether \( x_r \) represents the bond release length or some other point along the girder. This distinction is implicitly made when the strand patterns for the end of the girder and the centerline are defined. If the number of strands at the end of the member are fewer than at the centerline, then the difference represents the number of bond released strands, and their bond release length is assumed equal to \( x_r \) (if the number of strands at the end of the member is greater than the number of strands at the centerline, an error has been made). Three different potentially critical load combinations are used to calculate the stresses at point \( x_r \). The first loading is only the member weight (Mox) acting in conjunction with the full strand pattern.
under the initial prestress (fpi). The second loading is the member weight combined with the dead weight of the members framing into the girder, and the weight of the wet slab (Mox + Mdcx). For this load stage, the reduced strand pattern is used under the effective prestress (fpe). The third loading consists of all the service loads (Mox + Mdcx + Mdccx + Mlx) acting in conjunction with the reduced strand pattern under the effective prestress (fpe). It is, however, recommended that the stresses still be checked in the region of the first gap, should the bond release length not coincide with this region. This can be done by temporarily setting xr equal to the appropriate length and then checking only those stresses which result form the strand pattern that would actually be encountered at the chosen point. It should be kept in mind that the cross-sectional properties used for calculating the stresses at point xr do not include the contribution of the stubs and therefore may be conservative when xr falls within a stub region.

The next nine user-defined variables all pertain to the material used in the construction of the girder (Section A.6.2). The first three variables $fc$, $fci$, and $fcc$ define the strengths of the precast concrete, the green precast concrete, and the cast-in-place concrete respectively. The fourth variable $fpu$ define the tensile strength of the prestressing strands. The fifth variable $Ds$ defines the diameter of the strands, and the sixth variable $R$ defines the anticipated prestress losses in terms of a percentage of the initial prestress force. The next two variables $Size$ and $Grade$ define the bar size and yield stress of the supplementary top reinforcement. The last variable $Kc$ defines the allowable tensile stresses to be used under service loads for the bottom fibers of the girder. The ACI code, Section 18.4, specifies the
allowable extreme fiber stress in tension in the pre-compressed tensile zone to be either $6\sqrt{f_c}$ or $12\sqrt{f_c}$. If the latter value is used, immediate and long-term deflections must account for a cracked section and that cover requirements must comply with Section 7.7.3.2 of the ACI Code. In the spreadsheet, the variable $K_c$ is a factor applied to $\sqrt{f_c}$ and therefore should be assigned a value of either 6 or 12, but the decision is up to the designer. If a value of 12 is chosen, the spreadsheet will automatically calculate deflections using a bilinear moment deflection relationship, but satisfaction of the cover requirements of ACI 7.7.3.2 must be checked by the designer.

Step 4 - Estimation of the required prestress force and steel eccentricity using the Magnel diagram

This step provides the designer with two important items of information which can significantly improve the efficiency of the design process. This information is conveyed by way of a Magnel diagram presented in the MAGNEL DIAGRAM section (Section A.7). The first, and perhaps most important, item of information which can be obtained is an idea of whether there is any combination of prestress force and steel eccentricity which, if obtainable, could make the given member, under the given loads, satisfy all the allowable stress limits at the critical regions. The second piece of information which can be obtained is an idea of what combination of prestress force and steel eccentricity should be used as a first try in the design process, provided there is a possible combination. However, the appropriate prestress force for any chosen strand pattern is automatically calculated in Section A.10 and applied to the chosen strand pattern.
The Magnel diagram is generated by rewriting the stress equations for each of the critical points along the member so that equations are in terms of eccentricity as a function of the inverse of the prestress force. Allowable stresses (calculated in Section A.9.1) are then substituted into these equations. Plotting these equations results in a series of lines, each line defining a boundary for the limiting steel eccentricity that can be used with a given prestress force. These boundary lines can be separated into two categories. The first category is made up of those lines which are a function of limiting tensile stresses at the top of the member or limiting compressive stresses at the bottom of the member for the initial loading stage. The second category is made up of those lines which are a function of the limiting compressive stresses at the top of the member or the limiting tensile stresses at the bottom of the member for the service load stage. For the first category, the allowable eccentricity for a given prestress force can be at or below the boundary line on the Magnel diagram. For the second category, the allowable eccentricity for a given prestress force can be at or above the boundary line on the Magnel diagram. If there is a combination of prestress force and steel eccentricity which would satisfy the allowable stress criteria, then there will be a region in diagram where the two categories of boundary lines do not cross. The boundary lines associated with the service loading stage will form the lower border of this region and those associated with the initial loading stage will form the upper border of the region. It is the goal of the designer to provide a prestress force and eccentricity that falls within this region.

If a region defining possible combinations of prestress force and steel eccentricity is not present on the Magnel diagram, then some or all of the dimensions defining the cross-
section of the member will have to be adjusted until a region appears (see Step 2). It should be realized, however, that just because a region exists, it still may be impossible to actually achieve a combination that falls within the region due to the physical limitations of strand placement and allowable tendon stress. So, although a region may exist, it still may be necessary to adjust some or all of the cross-section dimensions.

Although a boundary line is generated which relates to the limiting tensile stresses at the top of the member for the initial stages of loading, it is often ignored since supplementary top steel will be provided in those regions where the allowable tensile stress is exceeded.

For convenience, a horizontal and vertical line is automatically plotted on the Magnel diagram, the intersection of which indicates the current steel eccentricity and prestress force. This feature is particularly helpful for quickly ascertaining how far the actual design is from converging on a design which will satisfy the allowable stress criteria. It must be remembered, however, that meeting the allowable stress criteria does not necessarily mean the member will have adequate strength to support the ultimate load.

Step 5 - Definition of the required strand pattern

This step involves the definition of the strand pattern at both the mid-span of the member and the end of the member, and is carried out in the FLEXURAL DESIGN SPECIFICATION section (Section A.6.3). Since only straight strands are considered, providing a means of specifying a strand pattern at the girder ends which is different from the strand pattern at the centerline is done for the purpose of defining bond released strands.
The definition of the strand pattern is carried out in the tables found in Sections A.6.3. (c) and (d). The first table is for the definition of the strand pattern at the centerline and the second table is for the definition of the strand pattern at the end of the member.

Defining the strand pattern is made as simple as possible by predefining certain values within the spreadsheet such as the lateral concrete cover for the strands and the spacing between each row and column of strands. Based on the dimensions commonly used by industry for strand placement in inverted tees, these values are set to 3.0 inches and 2.0 inches respectively. By way of these predefined values, the maximum number of strands per row can be automatically calculated for the given member geometry. This value is displayed above the first table. Another key value which is automatically calculated is maximum allowable prestress force which can be delivered by the chosen strand pattern and given member dimensions. The maximum allowable prestress force is determined by rewriting the stress equations for the critical points such that they provide the limiting force as a function of the chosen steel eccentricity and the allowable stress for the given point (Section A.10). The maximum permissible value is then translated into an effective tendon stress. This stress, however, is limited by the ACI Code. Therefore, the maximum allowable prestress force may be controlled by the allowable tendon stress.

The designer needs to provide three key items of information in these tables in order to completely define the strand pattern, the rest of the information is generated automatically. The first item of information is the desired cover between the bottom of the member and the first row of strands. This information is entered in the shaded cell at the top of the second
column of first table. The recommended value is 3.0 inches. The second item of information needed is the desired number of strands to be used in each row at the centerline of the member. This information is entered in the shaded cells of the fourth column of the first table (Section A.6.3 (a)). The third item of information needed is the desired number of strands to be used in each row at the end of the member. This information is entered in the shaded cells of the fourth column of the second table (Section A.6.3 (d)).

It is up to the designer, however, to insure that the number of strands specified for a given row does not exceed the maximum allowable number of strands. Although the allowable number of strands is displayed above the first table, and is automatically adjusted as the geometry of the member changes, no warning is given if this number is exceeded.

If bond released strands are not used, then the strand pattern at the ends must be made identical to that specified for the centerline. If bond released strands are used, the number of strands to be released is specified by defining the strand pattern at the end of the member such that it is the same as the pattern at the centerline, less the tendons which are to be released.

The easiest way to quickly arrive at a suitable strand pattern is to first assume a trial pattern. Next, refer to the Magnel diagram and see where the lines representing the chosen steel eccentricity and prestress force fall in relation to the allowable design region (it is assumed that a possible design region has already been identified on the Magnel diagram in Step 4). If they do not cross within the design region, but are close to it, then perhaps some minor adjustments of the pattern may be made which would result in a suitable combination of prestress force and steel eccentricity. If they are not at all close to the design region, it may be necessary to adjust some of the cross-section dimensions (Section A.5.2).
Step 6 - Check of the strength, critical stresses, and deflections

Once the cross-section dimensions and prestressing specifications have been adjusted to the point where, according to the Magnel diagram, the member stresses appear to fall within the allowable limits, the next step is to check the member for strength, allowable stresses, and deflections. These formal checks are presented in the FLEXURAL STRENGTH, STRESS, AND DEFLECTION CHECKS section (Section A.8). The method of checking any of the calculated values follows essentially the same format for either the strength, stress, or deflection checks. The spreadsheet automatically compares the given calculated value to the appropriate limit values and displays an OK adjacent to the calculated value if it is within an acceptable range relative to the limit values. If the calculated value is unacceptable, an NG (NG = No Good) is displayed in place of the OK.

Flexural Strength:

The nominal flexural strength of the girder is calculated for two critical stages of loading, the non-composite stage and the composites stage. For the non-composite stage, the cross-section used is that of a gap region, and the estimated tendon stress at failure for this stage is calculated in Section A.6.4 (a). The composite cross section used for the composite stage of loading is also taken as that of a gap region, and the estimated tendon stress at failure for this stage is calculated in Section A.6.4 (b). For both the non-composite and composite
stages, the tendon stress at failure is calculated by means of the approximate method given in the ACI Code, Section 18.7.2.

The nominal strengths are checked against the appropriate factored moments acting at the given stages of loading (Section A.8.1 (a) and (b)). If the strength is insufficient for either one of these load cases, it may still be helpful to follow through with a check of the critical stress regions. Checking the critical stress regions may give a better idea of where the member is deficient, providing some indication of what changes should be tried first. If any of the critical stress locations exceed the allowable stresses for the initial loading stage, then it is likely that the dimensions of the cross-section will have to be adjusted. If none of the allowable stress limits are violated, or if the allowable stresses are violated for service loads only, it may be wise to try rearranging the strand pattern or adding one or more strands before making any other adjustments. This may adequately increase the nominal strength, without violating any of the allowable stresses. If this does not work, the dimensions of the cross-section will have to be adjusted, and perhaps the strand pattern as well. Increasing any of the cross-section dimensions should be avoided when possible, since this adds to the size and weight of the member.

**Critical Stresses:**

If the nominal flexural strength requirements are met, then the next step is to check the stresses at the critical regions for the different stages of loading. The critical stresses, along with their code specified allowable limits, are summarized in Section A.8.2. The calculation of these critical stresses and the allowable stress limits are carried out in Section A.9.1. For each critical region, the stresses are checked at the outer most fibers of the given
precast section. For the composite stages of loading, the stresses are also checked at the top and bottom of the cast-in-place slab.

The first load stage that is checked is immediately after prestress transfer, where the non-composite girder is acted on by only the initial prestress force (Pi) and the self-weight of the girder (Wo). Three critical stress regions are checked for this load stage (Section A.8.2). For this load stage, however, the allowable tension stress used for comparison with the actual stresses is automatically adjusted to reflect the tension force carried by the supplementary top steel specified for the given region in Section A.9.4. The first critical region checked is at the girder ends. The stresses are actually calculated at a distance of 50 strand diameters in from the end of the girder, which is the theoretical point of full prestress transfer, as specified in Section 2.2.2 of the PCI Design Handbook. This distance is automatically defined by the variable xe, and is used to calculate the self-weight moment (Moe) occurring at this critical region (Section A.6.1 (c)). The equations used for calculating these end stresses account for the presence of bond released tendons, should bond releasing be used in the design. The stress equations for the end region, in contrast to all the other stress equations, account for the contribution of the stubs to the section properties. It was found that disregarding the stubs in this critical region leads to an overly conservative estimate of the actual stresses which unnecessarily controls the design. The second critical region that is checked is at the user defined point xr under the self-weight moment at point xr (Mox), calculated in Section A.6.1 (b). The equations for the actual stresses at point xr for this load stage assume the presence of the full strand pattern regardless of whether bond released tendons are used in the design or not. This assumption reflects the most critical situation, should xr be used to define
the transition point for the bond released tendons, as is recommended in Step 3. The third, and final region that is checked for this stage of loading is at the mid-span of the girder.

The next load stage that is checked occurs during construction, before the onset of composite action. The loading present at this stage results from the self-weight of the girder, the dead weight of the members framing onto the girder, and the weight of the wet cast-in-place slab, \((W_o + W_{dc})\). Two critical regions are checked for this stage of loading (Section A.8.3). The first is at the user defined point \(x_r\). The stress equations used for this load stage, at point \(x_r\), are set up to account for bond released tendons, should bond releasing be used in the design, and the loads used are \(M_{ox}\) and \(M_{dcx}\) calculated in Section A.6.1 (b). The second critical region checked is for the mid-span of the girder.

The final load stage that is checked is the full service load stage \((M_o + M_{dc} + M_{dce} + M_l)\) where the member is acting compositely with the cast-in-place slab (Section A.8.4). The first region check is at the user defined point \(x_r\). Similar to the previous load stage, the stress equations for point \(x_r\) used for this load stage are set up to account for bond released tendons, should bond releasing be used in the design.

If any of the member stresses violate their corresponding allowable limits, adjustments of either the strand pattern, or the dimensions of the cross-section, or both, will have to be made. However, it is assumed that prior to beginning this step of the design process, it was established by way of the Magnel diagram that the design will most likely not violate the allowable stress limits. As a result, checking the stresses usually involves nothing more than verifying that an OK appears adjacent to each of the actual member stress values for each of

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the critical load stages.

**Mid-span Deflections:**

The mid-span deflections, which would affect non-structural elements connected to the floor after the onset of composite action, are checked against the limit ACI suggested limit of L/480 (Section A.8.5). If an allowable bottom tension under service loads of $6\sqrt{f_{c}}$ is specified in Section A.6.2 by means of the user defined variable $K_c$, then the short-term deflection calculations are based on an elastic, uncracked section. However, if an allowable bottom tension of $12\sqrt{f_{c}}$ is specified, then short-term deflections are calculated using the bilinear moment method presented in the PCI Design Handbook, Section PCI 4.6.3. Long-term deflections are estimated by applying the suggested long-term deflection factors, suggested in Section 4.6.5 of the PCI Design Handbook, to the appropriate components of the short-term deflections. The deflections affecting the non-structural elements are taken as those due to the superimposed dead and live load combined with the time dependent portions of the deflections resulting from the prestress force and all other dead loads. All deflection calculations are presented in Section A.9.5.

**Step 7 - Shear reinforcement design for the stub regions**

This step of the design procedure involves the definition of only four variables which are found in the `SHEAR REINFORCEMENT DESIGN FOR THE STUB REGIONS` section of the spreadsheet (Section A.11).

The first user defined variable $x$ defines the section of interest, measured from the end,
for calculating the vertical shear design forces (Section A.11.1 (a)). It is recommended, however, for efficiency of the design and fabrication operations, the shear reinforcement for all of the stubs be made the same. The first stub experiences the maximum shear forces, and therefore should be the controlling stub for the shear design. The variable $x$ therefore should be set equal to half the depth of the lower flange ($h_{2c}/2$). This recommendation for $x$ is based on the commentary, Section R11.1.3 of the ACI Code, with the assumption that the loads are applied to the girder through the lower flange.

The second variable $u_c$ defines the concrete coefficient of friction which relates to the cast-in-place concrete interface with the tops of the stubs (Section A.11.1 (b)). For the stub girder design, the cast-in-place concrete is cast against hardened concrete which is assumed to be free of laitance, and intentionally roughened to a full amplitude of approximately $6.4$ mm (1/4 in). Therefore, for normal weight concrete, the suggested coefficient of friction is 1.00, according to Section 11.7.4 of the ACI Code.

The third variable $#st$ defines the bar size to be used for the stirrups which serve as both vertical shear reinforcement and horizontal shear reinforcement (Section A.11.1 (c)). However, only #3 through #6 bars can be used. The spreadsheet automatically provides the steel area of one stirrup for the given bar size (only bars #3 through #6 have been programmed into the spreadsheet). It is recommended, however, that the stirrups be no greater than #5 bars. This suggestion limit on stirrup size is based on research conducted at the University of Washington which, among other things, investigated the development of the yield strengths of hoop stirrups for different combinations of bar sizes, concrete strengths and slab thicknesses [Mattock 1987].
The fourth variable $f_{yr}$ defines the yield stress of the stirrups, but it is recommended that the yield stress not be greater than 344.75 MPa (50 ksi) (Section A.11.1 (c)). These suggested limitations are also based on the research conducted at the University of Washington, sighted above [Mattock 1987].

After the above variables have been defined, and assuming Steps 1 through 6 have been carried out successfully, the remainder of the stub shear reinforcement design is carried out by the spreadsheet according to the ACI Code. First, the concrete shear strength is calculated (Section A.11.3). Next, the vertical shear reinforcement requirements are determined, resulting in a suggested stirrup spacing necessary to provide the required shear strength (Section A.11.4 (a)). However, in most of the stub girder designs performed to date it was found that, the maximum stirrup spacing allowed by the ACI Code will control. After a suggested spacing has been determined, a check is made to verify that the minimum shear steel requirements of the ACI Code are met (Section A.11.4 (b)). Next, the shear force to be carried by the steel reinforcement alone is checked against the limits imposed by the ACI Code (Section A.11.4 (c)). Finally the nominal shear strength of the girder is checked against the design shear force associated with the section of interest defined previously by the variable $x$ (Section A.11.4 (d)).

After the reinforcement requirements for vertical shear have been determined, the required stirrup spacing for the horizontal shear acting at the stub and cast-in-place slab interface is determined. Since the reinforcement for the shear at this interface is provided by extending the stirrups through the top of the stubs, and into the cast-in-place slab, the stirrup spacing used for the final design is the minimum spacing required for either vertical shear or
horizontal shear. For the purpose of design and fabrication efficiency, the controlling spacing is also assumed to remain constant throughout the length of the each stub.

The horizontal force to be accommodated at the interface is taken as the change in the compressive force in the cast-in-place flange across a given segment of the member. The magnitude of the compressive force in the flange at any section is determined by dividing the moment acting on the girder, at the given section, by the available moment arm. Since the change in compressive force over a given segment is directly related to the change of moment across the segment, the critical horizontal shear transfer region occurs over the first stub, where the change in moment (i.e. the shear) is the greatest. The horizontal force to be taken by the first stub is calculated using the factored moment which acts at a distance from the end of the member equal to the length of the first stub plus the adjacent gap. The horizontal force is calculated at this point rather than at end of the first stub for the reason that, in the gap region, the cast-in-place flange is unable to transfer any horizontal forces to the girder. As a result, the stub must be able to accommodate the additional horizontal forces which develop across the length of the adjacent gap (Section A.11.5 (a)).

The required horizontal shear reinforcement is calculated using two different methods. The first method is called the Shear Strength Method (Section A.7.5 (a)), and the second is called the Shear Friction Method (A.11.5. (b)). The shear friction method tends to be rather conservative, so, when applicable, the spacing requirements based on the shear strength method are used. However, the ACI Code limits the horizontal force that can be carried according to the shear strength method. If those limitations are exceeded, the shear friction method is used to control the required spacing.
The results of the shear reinforcement design process for the stub region can be found in Section A.11.2, and in the stub girder design summary in Section A.14.6.

Step 8 - Shear reinforcement design for the gap regions:

This step is carried out in **THE SHEAR REINFORCEMENT DESIGN FOR THE GAP REGIONS** section (Section 12) and involves the definition of only one variable. The shear reinforcement design for the gap region is essentially the same as that for the stub region described in Step 7. The difference is that, for the gap region, there is no interface between the girder and the cast-in-place slab, so there are no horizontal shear considerations.

The only user defined variable is $x_g$ which, similar to $x$ in the previous step, defines the section of interest for calculating the vertical shear design forces (Section 12.1 (a)). It is also recommended for the gap regions that the shear reinforcement spacing be the same for all gaps, and the design be controlled by the gap which experiences the greatest shear forces. Therefore, $x_g$ should be set equal to the distance to the beginning of the first gap. For the purpose of simplifying the design and fabrication of the stub girder, the gap region shear reinforcement will also use the same size and grade stirrups as defined in Step 7 for the stub regions. The shear reinforcement design calculations for the gap regions, which are the same as those carried out for the vertical shear reinforcement of the stub regions, are presented in Section A.11 of the design aid. The results of these calculations for the gap regions are presented in Section A.12.2 and in the stub girder design summary Section A.14.6.

Step 9 - Torsion reinforcement design
This step is carried out in the TORSION REINFORCEMENT DESIGN section (Section A.13), and requires the definition of only one variable.

The torsional moment arises during the phase of construction when only one side of the girder is loaded. This stage of loading is only temporary and consists of the weight of the double tees and the construction live load (although it is reasonable to use the wet weight of the slab in place of a construction live load). The moment arm for this torsional load is taken as the distance from the centerline of the girder to the theoretical point of application of the double tee stem loads on the lower flange of the girder. This theoretical point of load application is taken as 1/4 of the bearing surface (b2c * 1/4) in from the edge of the lower flange.

The variable xt defines the critical section for the investigation for the calculation of the torsional loads (Section A.13.1 (a)). For the stub girder, it is recommended to take the beginning of the first gap as the critical torsion section since this is the smallest cross-section which will experience the highest combination of shear and torsion. However, for the stub girder, it is unlikely that the shear loading will have an impact on the design. The calculations are set up to use the dimensions associated with the gap region since it is assumed that the first gap will be specified as the critical section. Therefore, if xt is defined to investigate a cross-section that does not fall in a gap region, the spreadsheet will have to be modified in order to handle the required calculations.

The nominal concrete torsional strength of the chosen cross-section is calculated in Section A.13.2. Experience with stub girder designs indicates that the nominal torsional strength of the concrete at the first gap will be more than adequate to carry the applied load.
torsional moments. As a result, additional torsional reinforcement will not be required to carry the torsional moments. In light of this experience, the spreadsheet has only been set up to calculate the nominal torsional strength of the concrete alone, and compare this value to the applied torsional moment. The spreadsheet has not been set up to calculate the required reinforcement, should reinforcement be needed. If this is the case, the spreadsheet will have to be modified or the calculations will have to be done by hand.

The results of the comparison of the nominal torsional strength of the concrete to the applied torsional moment is presented in Section A.13.2 (b) and the stub girder design summary Section A.14.6.
A.4. DESIGN LOADS

Member: Stub Girder 34 ST 38-18-20

A.4.1. Initial Data

a) Tributary members and dimensions

Column spacing in girder direction: XBay 32 ft
Column spacing in floor beam direction: YBay 40 ft

Double tee member used: Floor 20 DT

# of floor members on one side of the girder: Nfloor 4

CIP topping thickness (in): ts 2

Girder tributary area (ft^2): Atr 1.280

b) Tributary loads

Double tee self weight per sq. ft. (psf): Qo 47
Superimposed dead load (psf): Qd 15.0
Superimposed live load (psf): Ql 50.0

Distributed load of CIP topping (psf): Qcip 25

A.4.2. Double Tee Stem Loads

a) Stem load due to DT self weight and CIP topping:

Psdc = Atr * (Qo + Qcip) / (Nfloor * 4)

Psdc = 5.76 k

b) Stem load due to superimposed dead loads only:

Psdcc = Atr * (Qd) / (Nfloor * 4)

Psdcc = 1.20 k

c) Stem load due to superimposed live loads:

Psi = Atr * (Ql) / (Nfloor * 4)

Psi = 4.00 k

A.4.3. Approximate Stub Girder Uniform Load Resulting From Stem Loads

a) Uniform load due to DT self weight and CIP topping:

Wdc = Nfloor * 4 * Psdc * 1000 / L

Wdc = 3021.64 k-ft

b) Uniform load due to superimposed dead loads only:

Wdccc = Nfloor * 4 * Psdccc * 1000 / L

Wdccc = 629.51 k-ft

c) Uniform load due to superimposed live loads:

Wl = Nfloor * 4 * Psi * 1000 / L

Wl = 2098.36 k-ft
A.5. GEOMETRIC MEMBER DEFINITION

Member Stub Girder BC ST h - h1c - h2c

A.5.1. CIP Composite Compression Flange Dimensions

| Slab thickness b/w stubs (in): | tc | 6.00 |
| Slab depth above the stubs (in): | ta | 4.00 |

Width of CIP compression flange available either side of girder stub

( = amount of DT flange removed) \( W_{cip} \) 24.00 in

A.5.2. Stub Girder Dimensions:

| Member length (ft): | L | 30.50 ft |
| Total PC member depth (in): | hc | 38.00 |
| Stub height (in): | h1c | 18.00 |
| Bottom flange depth (in): | h2c | 20.00 |
| Total member width (in): | bc | 34.00 |
| Stub width (in): | b1c | 22.00 |
| Bottom flange width (in): | b2c | 6.00 |
| \( h_{1c} + ta \) (used for calculations): | \( Y_{c} \) | 22.00 |

Effective transposed flange width:

\( b_{trc} \) 59.00

(smaller of: \( 16\times tc + b1c, L/4 \), or \( 2 \times W_{cip} + b1c, \) times \( n \))

[ACI 8.10.2]

| Space between stubs (gaps) (in): | Gap | 34.00 |
| Length of stubs (in): | Stub | 66.00 |
| Number of gaps in beam: | Ngap | 3 |
| Number of stubs in beam: | Nstub | 4 |

Average member weight per foot: \( W_t \) 1,005.9 lbs/ft

Total member weight (lbs) \( W_{tot} \) 30,679.2 lbs

A.5.3. Basic Section Properties

Section-1: (non-comp. at stubs)

| \( A_{1c} \) | 1,076.0 |
| \( C_{11c} \) | 21.01 |
| \( I_{1c} \) | 123,703 |

Section-2: (non-comp. at gaps)

| \( A_{2c} \) | 680.0 |
| \( C_{12c} \) | 10.00 |
| \( I_{2c} \) | 22,667 |

A-27
A.5.4. Detailed Summary of All Section Properties

a) Moments of Inertia \((in^4)\)

<table>
<thead>
<tr>
<th></th>
<th>(Ics)</th>
<th>(Iccs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-composite moment of inertia at stubs:</td>
<td>123,702.6</td>
<td>241,603.9</td>
</tr>
<tr>
<td>Composite moment of inertia at stubs:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>(Icg)</th>
<th>(Iccg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-composite moment of inertia at gaps:</td>
<td>22,666.7</td>
<td>202,329.6</td>
</tr>
<tr>
<td>Composite moment of inertia at gaps:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

b) Centroidal axis (CA) measurements for all cross-sections \((in)\)

**Stubs:**

<table>
<thead>
<tr>
<th></th>
<th>(C1cs)</th>
<th>(C2cs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dist. from top of PC mem. to non-comp. CA:</td>
<td>21.01</td>
<td>16.99</td>
</tr>
<tr>
<td>Dist. from bot. of PC mem. to non-comp. CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from top of PC mem. to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist. from bot. of PC mem. to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from top of CIP slab to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from bot. of CIP slab to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tendon eccentricity for the non-comp. CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tendon eccentricity for the composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reduced tendon eccentricity for the non-comp.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>* (eics) will differ from (ecs) only if there are bond released tendons.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>(ecs)</th>
<th>(eccs)</th>
<th>(eics)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dist from top of PC mem. to composite CA:</td>
<td>13.57</td>
<td>18.41</td>
<td>13.57</td>
</tr>
<tr>
<td>Dist from bot. of PC mem. to composite CA:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from top of CIP slab to composite CA:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from bot. of CIP slab to composite CA:</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Gaps:**

<table>
<thead>
<tr>
<th></th>
<th>(C1cg)</th>
<th>(C2cg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dist. from top of PC mem. to non-comp. CA:</td>
<td>10.00</td>
<td>10.00</td>
</tr>
<tr>
<td>Dist. from bot. of PC mem. to non-comp. CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from top of PC mem. to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist. from bot. of PC mem. to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from top of CIP slab to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from bot. of CIP slab to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tendon eccentricity for the non-comp. CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tendon eccentricity for the composite CA:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>(ecg)</th>
<th>(eccg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dist from top of PC mem. to composite CA:</td>
<td>6.58</td>
<td>15.50</td>
</tr>
<tr>
<td>Dist from bot. of PC mem. to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from top of CIP slab to composite CA:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dist from bot. of CIP slab to composite CA:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**c) Cross-section areas \((in^2)\)**

<table>
<thead>
<tr>
<th></th>
<th>(Acs)</th>
<th>(Accs)</th>
<th>(Acs)</th>
<th>(Accg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-composite area at stubs:</td>
<td>1,076.0</td>
<td>1,371.0</td>
<td>680.0</td>
<td>975.0</td>
</tr>
<tr>
<td>Composite area at stubs:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Non-composite area at gaps:</td>
<td>1,076.0</td>
<td>1,371.0</td>
<td>680.0</td>
<td>975.0</td>
</tr>
<tr>
<td>Composite area at gaps:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

A-28
A.6. FLEXURAL DESIGN SPECIFICATIONS

Member: Stub Girder 34 ST 38-18-20

A.6.1. Design Loads

a) Service loads

<table>
<thead>
<tr>
<th>Loads</th>
<th>(plf)</th>
<th>Moments</th>
<th>(k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wo</td>
<td>1005.9</td>
<td>Mo</td>
<td>117.0</td>
</tr>
<tr>
<td>Wdc</td>
<td>3021.6</td>
<td>Mdc</td>
<td>351.4</td>
</tr>
<tr>
<td>Wdcc</td>
<td>629.5</td>
<td>Mdcc</td>
<td>73.2</td>
</tr>
<tr>
<td>WI</td>
<td>2098.4</td>
<td>MI</td>
<td>244.0</td>
</tr>
<tr>
<td></td>
<td>6755.4</td>
<td></td>
<td>785.5</td>
</tr>
</tbody>
</table>

b) Loads at chosen critical point of interest for flexural analysis

Location of \( x_r \) measured from end (in): \( x_r \) = 86.00

Moments calculated at point \( x_r \):

<table>
<thead>
<tr>
<th>Moments</th>
<th>(k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mo</td>
<td>69.15</td>
</tr>
<tr>
<td>Mdcx</td>
<td>207.74</td>
</tr>
<tr>
<td>Mdccx</td>
<td>43.28</td>
</tr>
<tr>
<td>Mix</td>
<td>144.26</td>
</tr>
</tbody>
</table>

c) Loads at theoretical point of prestress transfer:

Point of prestress transfer 50 * ds (ft):

\( x_e \) = 2.083

Self weight moment at transfer point (k-ft):

\( \text{Moe} \) = 29.77

A.6.2. Material Specifications

Precast Concrete (psi): \( f_c \) = 5000
Green Precast Concrete (psi): (0.7 to 0.75) \( f_{ci} \) = 3500
CIP (psi): \( f_{cc} \) = 3500

Young's Modulus for PC (ksi): \( E_c \) = 4074
Young's Modulus for CIP (ksi): \( E_{cip} \) = 3409
Young's Modulus for prestressing steel (ksi): \( E_p \) = 28500

Prestressing Steel (ksi): \( f_{pu} \) = 270
Strand diameter, assuming 7-wire type (in): \( D_s \) = 0.500
Prestress Loss:

\( R \) = 0.15

Supplementary tensile reinforcement:

Bar size from 3 to 6: (i.e. enter 5 for #5 bar) Size = 5
Yield stress of reinforcing steel (ksi): Grade = 50.0

Allowable tensile stress \( K_c \times \sqrt{f_c} \) (\( K_c = 6 \) or 12) \( K_c \) = 12

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A.6.3. **Member Properties For Flexural Analysis**

The flexural analysis and design counts on composite action to carry the design loads. For the non-composite stages, the section properties for the entire member are taken as those of only the continuous bottom flange. For the composite stages, the section properties for the entire member are taken as those of only the continuous bottom flange with the added contribution of the transposed composite slab. Only when determining the critical member end stresses is the contribution of the stubs considered.

a) **Non-composite member properties for flexural analysis**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of precast member (in^2)</td>
<td>( A_c )</td>
</tr>
<tr>
<td>PC Moment of inertia (in^4)</td>
<td>( I_{c} )</td>
</tr>
<tr>
<td>PC member depth (in)</td>
<td>( h )</td>
</tr>
<tr>
<td>Top to centroid (in)</td>
<td>( C_{1c} )</td>
</tr>
<tr>
<td>Bottom to centroid (in)</td>
<td>( C_{2c} )</td>
</tr>
<tr>
<td>Top Section Modulus (in^3)</td>
<td>( S_{1c} )</td>
</tr>
<tr>
<td>Bottom Section Modulus: (in^3)</td>
<td>( S_{2c} )</td>
</tr>
</tbody>
</table>

b) **Composite member properties for flexural analysis**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of composite member (in^2)</td>
<td>( A_{cc} )</td>
</tr>
<tr>
<td>CIP Moment of inertia (in^4)</td>
<td>( I_{cc} )</td>
</tr>
<tr>
<td>Effective Slab at location of beam (in)</td>
<td>( t_e )</td>
</tr>
<tr>
<td>Full CIP depth at location of beam (in)</td>
<td>( t )</td>
</tr>
<tr>
<td>Full depth of composite member (in)</td>
<td>( h_{cc} )</td>
</tr>
<tr>
<td>Top of PC member to centroid (in)</td>
<td>( C_{1cc} )</td>
</tr>
<tr>
<td>Bottom of PC member to centroid (in)</td>
<td>( C_{2cc} )</td>
</tr>
<tr>
<td>Top of CIP slab to centroid (in)</td>
<td>( C_{3cc} )</td>
</tr>
<tr>
<td>Bottom of CIP slab to centroid (in)</td>
<td>( C_{4cc} )</td>
</tr>
<tr>
<td>PC top section modulus ( I_{cc} / C_{1cc} )</td>
<td>( S_{1cc} )</td>
</tr>
<tr>
<td>PC bottom section modulus ( I_{cc} / C_{2cc} )</td>
<td>( S_{2cc} )</td>
</tr>
<tr>
<td>Section modulus to top of slab ( I_{cc} / C_{3cc} )</td>
<td>( S_{3cc} )</td>
</tr>
<tr>
<td>Modular Ratio (E_{CIP}/ E_{c})</td>
<td>( n )</td>
</tr>
<tr>
<td>Composite tendon eccentricity (in)</td>
<td>( ecc )</td>
</tr>
</tbody>
</table>
c) Strand pattern at center line

Allowable number of strands per row with 2” spacing:
(Assumes 3 in. lateral cover)

<table>
<thead>
<tr>
<th>Rows From Bottom</th>
<th>Distance From Bottom</th>
<th>Distance from Precast CA</th>
<th>Dist. from CA Times #strand</th>
<th>Number of Strands /Row</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row #1</td>
<td>3.00</td>
<td>7.00</td>
<td>105.00</td>
<td>15</td>
</tr>
<tr>
<td>Row #2</td>
<td>5.00</td>
<td>5.00</td>
<td>20.00</td>
<td>15</td>
</tr>
<tr>
<td>Row #3</td>
<td>7.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Row #4</td>
<td>9.00</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Tendon eccentricity at centerline (in):

| e | 6.58 |

d) Strand pattern at member ends:

The end strand pattern is where the bond released tendons are defined. The bond release length for these tendons is defined in Sect. A-5.1 with the dual purpose variable $x_r$. If no tendons are to be bond released, the end pattern should made identical to the center-line pattern.

<table>
<thead>
<tr>
<th>Row From Bottom</th>
<th>Distance From Bottom</th>
<th>Distance from Precast NA</th>
<th>Dist. from NA Times #strand</th>
<th>Number of Strands /Row</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row #1</td>
<td>3.00</td>
<td>7.00</td>
<td>105.00</td>
<td>15</td>
</tr>
<tr>
<td>Row #2</td>
<td>5.00</td>
<td>5.00</td>
<td>20.00</td>
<td>4</td>
</tr>
<tr>
<td>Row #3</td>
<td>7.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Row #4</td>
<td>9.00</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Tendon eccentricity at member ends (in):

| ei | 6.58 |

e) Prestressing Steel Specifications

Initial prestress (ksi):
(Limit = 200
(Limit = 0.74 * fpu, [ACI 18.5])

| fpi | 193.5 |

<table>
<thead>
<tr>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>183.9</td>
<td>194.0</td>
</tr>
<tr>
<td>534.5</td>
<td>563.9</td>
</tr>
</tbody>
</table>

Total number of strands:

| Ns   | 19 |

Total area of prestress steel (in*2): 2.907

Number of bond released tendons at ends:

| Nrelease | 0 |

Tendon bond release length at ends (in):

| Lrelease | 0.00 |
A.6.4. Tendon Stress at Failure and Check of Reinforcement Ratio

a) Tendon stress at failure for non-composite stage

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Available compression flange width (in)</td>
<td>34.00</td>
</tr>
<tr>
<td>Dist. from outer compression fiber to Ap</td>
<td>16.56</td>
</tr>
<tr>
<td>Steel ratio Ap / (bg * dg)</td>
<td>0.0052</td>
</tr>
<tr>
<td>Ratio of fpe / fpu</td>
<td>0.609</td>
</tr>
<tr>
<td>Check for validity of ACI fps equation 18.3:</td>
<td>fpe/fpu &gt; 0.5 OK</td>
</tr>
</tbody>
</table>

Tendon stress at failure by ACI equation 18.3 (ksi):

- $f_{ps}$ = $f_{pu} \times (1 - 0.28/8 \times r_{pi} \times f_{pu}^{1 - f_{cc}})$

Reinforcement Index $w_{pi}$ [ACI 18.0]:

- $w_{pi} = A_p / (d_g \times b_{wg}) \times (f_{ps} / f_{cc})$

Upper limit on $w_{pi}$ [ACI 18.1]:

- $0.36 \times B = 0.288$

Check of upper limit on $w_{pi}$:

- $w_{pi} \leq 0.36 \times B$ OK

b) Tendon stress at failure for full composite section

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transp. effective compression width (in)</td>
<td>59.00</td>
</tr>
<tr>
<td>Dist. from outer compression fiber to Ap</td>
<td>38.58</td>
</tr>
<tr>
<td>Steel ratio Ap / (b*dp)</td>
<td>0.0013</td>
</tr>
<tr>
<td>Ratio of fpe / fpu</td>
<td>0.609</td>
</tr>
<tr>
<td>Check for validity of ACI fps equation 18.3:</td>
<td>fpe/fpu &gt; 0.5 OK</td>
</tr>
</tbody>
</table>

Tendon stress at failure by ACI equation 18.3 (ksi):

- $f_{ps} = f_{pu} \times (1 - 0.28/8 \times r_{p} \times f_{pu} / f_{cc})$

Reinforcement Index [ACI 18.0]:

- $w_{p} = A_p / (d_p \times b_{tr}) \times (f_{ps} / f_{cc})$

Upper limit on $w_{p}$ [ACI 18.1]:

- $0.36 \times B = 0.306$

Check of upper limit on $w_{p}$:

- $w_{p} \leq 0.36 \times B$ OK
A.7. MAGNEL DIAGRAM

Member: Stub Girder 34 ST 38-18-20

A.7.1. Magnel Diagram

A.7.2. Load and Member Specifications

Upper Kern (in): S2c/Ac
Lower Kern (in): S1c/Ac
Self wt. moment at point (xr):

A.7.3. Magnel Equations

a) Limits on (e) under non-composite action

\[ e_1 \leq \frac{Ics}{(Acs \cdot C1cs)} + \frac{(ftis \cdot Ics/C1cs + Mox \cdot 12)}{1/Pi} \]
\[ e_2 \leq -\frac{Ics}{(Acs \cdot C2cs)} + \frac{(-fci \cdot Ics/C2cs + Mox \cdot 12)}{1/Pi} \]
\[ e_{3a} \leq \frac{k2c + (fts \cdot S1c + Mo + Mdc)}{1/(1-R)} \cdot \frac{1}{1/Pi} \]
\[ e_{4a} \leq -\frac{k1c}{(fcs \cdot S2c + Mo + Mdc)} \cdot \frac{1}{1/(1-R)} \cdot \frac{1}{1/Pi} \]
\[ e_{3b} \geq \frac{k2c + (fts \cdot S1c + Mo + Mdc)}{1/(1-R)} \cdot \frac{1}{1/Pi} \]
\[ e_{4b} \geq -\frac{k1c}{(-fts \cdot S2c + Mo + Mdc)} \cdot \frac{1}{1/(1-R)} \cdot \frac{1}{1/Pi} \]

b) Limits on (e) under composite action

\[ e_5 \geq \frac{k2c + (fts \cdot S1c + (Mo + Mdc) + (S1c/S2cc) \cdot (Mdcc + Ml))}{1/(1-R)} \cdot \frac{1}{1/Pi} \]
\[ e_6 \geq -\frac{k1c + (-fts \cdot S2c + (Mo + Mdc) + (S2c/S2cc) \cdot (Mdcc + Ml))}{1/(1-R)} \cdot \frac{1}{1/Pi} \]
A.8. FLEXURAL STRENGTH, STRESS AND DEFLECTION CHECKS

Member: Stub Girder 34 ST 38-18-20

A.8.1. Nominal Strength Check

a) Strength check for non-composite loads 1.4 (Mo + Mdc)

Estimated compression block depth for non-composite member:

\[ ai = \frac{Ap \times fpsi}{0.85 \times fc \times bwg} \quad [ACI ~ 10.7.1] \]

<table>
<thead>
<tr>
<th>ai</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.90</td>
</tr>
</tbody>
</table>

Nominal moment strength of non-composite member:

\[ ObMni = Ap \times fpsi \times (dg - ai/2) \geq Mui \]

<table>
<thead>
<tr>
<th>Mui</th>
</tr>
</thead>
<tbody>
<tr>
<td>655.65</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ObMni</th>
</tr>
</thead>
<tbody>
<tr>
<td>750.60</td>
</tr>
</tbody>
</table>

b) Strength check for composite loads 1.4 (Mo + Mdc + Mdcc) + 1.7Ml

Estimated depth of compression block in cast-in-place slab:

\[ acip = \frac{Ap \times (fpcc)}{0.85 \times fcc \times btr} \leq tc \quad [ACI ~ 10.7.1] \]

<table>
<thead>
<tr>
<th>acip</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.33</td>
</tr>
</tbody>
</table>

Nominal Strength of composite member

\[ ObMn = 0.9 \times (Ap \times fps \times (dp - acip/2) + Ap \times fpc \times (dg - acip/2)) \geq Mu \]

<table>
<thead>
<tr>
<th>Mu</th>
</tr>
</thead>
<tbody>
<tr>
<td>1172.93</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ObMn</th>
</tr>
</thead>
<tbody>
<tr>
<td>2074.10</td>
</tr>
</tbody>
</table>

A-34
A.8.2. **Check Of Stresses In The Initial Non-Composite Stage**

The allowable tensile stresses (Sect. A.9.1) for initial conditions automatically adjusts to account for the available supplementary tensile steel (Sect. A.9.4). Actual stresses are calculated in Sect. A.9.2.

The term “Reduced Strands” indicates that, if there are bond released strands, the given calculation will automatically account for

a) **Initial stresses at member ends for (Moe) - Reduced Strands**

<table>
<thead>
<tr>
<th>Allowable Stress Range</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp.</td>
<td>Tens.</td>
<td></td>
</tr>
<tr>
<td>-2.100</td>
<td>30.000</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stress Location</th>
<th>Actual</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC Top</td>
<td>0.713</td>
<td>OK</td>
</tr>
<tr>
<td>PC Bottom</td>
<td>-1.522</td>
<td>OK</td>
</tr>
</tbody>
</table>

The initial end stresses are calculated based on the entire PC cross-section including the contribution of the stubs.

b) **Initial stresses at point \( x_r \) for (Mox)**

<table>
<thead>
<tr>
<th>Allowable Stress Range</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp.</td>
<td>Tens.</td>
<td></td>
</tr>
<tr>
<td>-2.100</td>
<td>30.000</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stress Location</th>
<th>Actual</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC Top</td>
<td>0.439</td>
<td>OK</td>
</tr>
<tr>
<td>PC Bottom</td>
<td>-2.094</td>
<td>OK</td>
</tr>
</tbody>
</table>

c) **Initial stresses at mid-span for (Mo)**

<table>
<thead>
<tr>
<th>Allowable Stress Range</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp.</td>
<td>Tens.</td>
<td></td>
</tr>
<tr>
<td>-2.100</td>
<td>30.000</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stress Location</th>
<th>Actual</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC Top</td>
<td>0.186</td>
<td>OK</td>
</tr>
<tr>
<td>PC Bottom</td>
<td>-1.841</td>
<td>OK</td>
</tr>
</tbody>
</table>
A.8.3. Check of Stresses In The Intermediate Non-Composite Stage

<table>
<thead>
<tr>
<th>Component</th>
<th>Allowable Stress Range</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Comp.</td>
</tr>
<tr>
<td>PC Member</td>
<td>-2.250</td>
</tr>
</tbody>
</table>

a) Intermediate stresses at point \( x_r \) for \((M_{ox} + M_{dcx})\) - Reduced Strands

<table>
<thead>
<tr>
<th>Stress Location</th>
<th>Actual</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC Top</td>
<td>-0.660</td>
<td>OK</td>
</tr>
<tr>
<td>PC Bottom</td>
<td>-0.994</td>
<td>OK</td>
</tr>
</tbody>
</table>

b) Stresses at mid-span for \((M_o + M_{dc})\)

\[
\text{Allowable tensile stresses in PC} = 12 \times \sqrt{f_c}
\]

<table>
<thead>
<tr>
<th>Stress Location</th>
<th>Actual</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC Top</td>
<td>-1.795</td>
<td>OK</td>
</tr>
<tr>
<td>PC Bottom</td>
<td>0.389</td>
<td>OK</td>
</tr>
</tbody>
</table>

A.8.4. Stress Check For The Composite Stage

<table>
<thead>
<tr>
<th>Component</th>
<th>Allowable Stress Range</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Comp.</td>
</tr>
<tr>
<td>PC Member</td>
<td>-2.250</td>
</tr>
<tr>
<td>CIP Slab</td>
<td>-1.575</td>
</tr>
</tbody>
</table>

a) Stresses at point \( x_r \) for \((M_{ox} + M_{dcx} + M_{dccc} + M_{ix})\):

\[
\text{Allowable tensile stress in PC} = 12 \times \sqrt{f_c}
\]

<table>
<thead>
<tr>
<th>Stress Location</th>
<th>Actual</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC Top</td>
<td>-0.993</td>
<td>OK</td>
</tr>
<tr>
<td>PC Bottom</td>
<td>-0.414</td>
<td>OK</td>
</tr>
<tr>
<td>Slab Top</td>
<td>-0.215</td>
<td>OK</td>
</tr>
<tr>
<td>Slab bottom</td>
<td>-0.168</td>
<td>OK</td>
</tr>
</tbody>
</table>
b) Stresses at mid-span for \((M_o + M_{dc} + M_{dcc} + M_l)\):

Allowable tensile stress in PC  \(= \ 12 \times \sqrt{f_c}\)

<table>
<thead>
<tr>
<th>Stress Location</th>
<th>Actual</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC Top</td>
<td>-2.154</td>
<td>OK</td>
</tr>
<tr>
<td>PC Bottom</td>
<td>0.745</td>
<td>OK</td>
</tr>
<tr>
<td>Slab Top</td>
<td>-0.363</td>
<td>OK</td>
</tr>
<tr>
<td>Slab bottom</td>
<td>-0.284</td>
<td>OK</td>
</tr>
</tbody>
</table>

A.8.5. **Mid-Span Deflection (In)**

Allowable  \(= \frac{L}{480}\)  [ACI 9.5.2.5]

Check of mid-span deflections affecting non-structural elements:

<table>
<thead>
<tr>
<th>Deflection</th>
<th>Actual</th>
<th>Allowable</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Y_{crit})</td>
<td>-0.68</td>
<td>0.76</td>
<td>OK</td>
</tr>
</tbody>
</table>

* Deflection calculations are presented in Sect. A-6.5.
A.9 FLEXURAL DESIGN AND ANALYSIS CALCULATIONS

Member: Stub Girder 34 ST 38-18-20

A.9.1. Allowable Stresses [ACI 18.4.1]

a) Allowable PC member stresses immediately after transfer.

<table>
<thead>
<tr>
<th>Compression: 0.6 x f'cl (ksi)</th>
<th>fcl</th>
<th>-2.100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension @ CL: 3 x sqrt(f'cl) (ksi)</td>
<td>filc</td>
<td>0.177</td>
</tr>
<tr>
<td>Tension @ Suprt: 6 x sqrt(f'cl) (ksi)</td>
<td>fils</td>
<td>0.355</td>
</tr>
</tbody>
</table>

b) Allowable PC member stresses at service conditions.

<table>
<thead>
<tr>
<th>Compression: 0.45 x f'c (ksi)</th>
<th>fcs</th>
<th>-2.250</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension: 12 x sqrt(f'c) (ksi)</td>
<td>fts</td>
<td>0.849</td>
</tr>
</tbody>
</table>

c) Allowable stresses in CIP Slab.

<table>
<thead>
<tr>
<th>Compression: 0.45 x f'cc (ksi)</th>
<th>fcss</th>
<th>-1.575</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension: 6 x sqrt(f'cc) (ksi)</td>
<td>fcss</td>
<td>0.355</td>
</tr>
</tbody>
</table>

A.9.2. Non-Composite Member Stresses

The term "Reduced Strands" indicates that, if there are bond released strands, the given calculation accounts for that condition.

a) Stresses at center span for initial conditions (Mo)

Top of member:

\[ f_{1t} = - \frac{P_i}{A_c} + \frac{(P_i \cdot e \cdot C_{1c})}{l_c} - \frac{(M_o \cdot C_{1c})}{l_c} \]

\[ f_{1t} = 0.186 \]

Bottom of member:

\[ f_{2t} = - \frac{P_i}{A_c} - \frac{(P_i \cdot e \cdot C_{2c})}{l_c} + \frac{(M_o \cdot C_{2c})}{l_c} \]

\[ f_{2t} = -1.841 \]
b) Stresses at member ends for (Moe) - Reduced Strands

These member end stress calculations take into account the contribution the stubs make to the section properties.

Top of member:

\[ f_{1e} = -\frac{P_e}{A_e} + \frac{(P_e \cdot e_c \cdot C_{1c})}{I_{cs}} - \frac{(Moe \cdot C_{1c})}{I_{cs}} \]

\[ f_{1e} = 0.713 \]

Bottom of member:

\[ f_{2e} = -\frac{P_e}{A_e} - \frac{(P_e \cdot e_c \cdot C_{2c})}{I_{cs}} + \frac{(Moe \cdot C_{2c})}{I_{cs}} \]

\[ f_{2e} = -1.522 \]

c) Stresses at point \( x_r \) for (Mox):

Top of member:

\[ f_{1x} = -\frac{P_i}{A_i} + \frac{(P_i \cdot e \cdot C_{1c})}{I_{c}} - \frac{(Mox \cdot C_{1c})}{I_{c}} \]

\[ f_{1x} = 0.439 \]

Bottom of member:

\[ f_{2x} = -\frac{P_i}{A_i} - \frac{(P_i \cdot e \cdot C_{2c})}{I_{c}} + \frac{(Mox \cdot C_{2c})}{I_{c}} \]

\[ f_{2x} = -2.094 \]

d) Stresses at point \( x_r \) for (Mox + Mdcx) - Reduced Strands

Top of member:

\[ f_{1xr} = -\frac{P_i}{A_i} + \frac{(P_i \cdot e_i \cdot C_{1c})}{I_{c}} - \frac{(Mox + Mdcx) \cdot C_{1c}}{I_{c}} \]

\[ f_{1xr} = -0.660 \]

Bottom of member:

\[ f_{2xr} = -\frac{P_i}{A_i} - \frac{(P_i \cdot e_i \cdot C_{2c})}{I_{c}} + \frac{(Mox + Mdcx) \cdot C_{2c}}{I_{c}} \]

\[ f_{2xr} = -0.994 \]

e) Stresses at mid-span for (Mo + Mdc):

Top of PC member:

\[ f_{1s} = -\frac{P_e}{A_c} + \frac{(P_e \cdot e \cdot C_{1c})}{I_{c}} - \frac{(Mo + Mdc) \cdot C_{1c}}{I_{c}} \]

\[ f_{1s} = -1.795 \]

Bottom of PC member:

\[ f_{2s} = -\frac{P_e}{A_c} - \frac{(P_e \cdot e \cdot C_{2c})}{I_{c}} + \frac{(Mo + Mdc) \cdot C_{2c}}{I_{c}} \]

\[ f_{2s} = 0.389 \]
A.9.3. Composite Critical Stresses:

a) Stresses at point \( x_r \) for \((M_\text{o} + M_{dc} + M_{dcc} + M_{lx})\) - Reduced Strands

Top of member:

\[
\sigma_{1cc} = \frac{P_{ee}}{A_c} + \frac{(P_{ee} * e * C_{1c})}{I_c} - (M_\text{o} + M_{dc})C_{1c}/I_c - (M_{dcc} + M_{lx})C_{1cc}/I_c
\]

\[
\sigma_{1cc} = -0.993
\]

Bottom of member:

\[
\sigma_{2cc} = \frac{P_{ee}}{A_c} - \frac{(P_{ee} * e * C_{2c})}{I_c} + (M_\text{o} + M_{dc})C_{2c}/I_c + (M_{dcc} + M_{lx})C_{2cc}/I_c
\]

\[
\sigma_{2cc} = -0.414
\]

Top of CIP slab:

\[
\sigma_{3cc} = -(M_{dc} + M_{lx})C_{3cc} x n / lcc
\]

\[
\sigma_{3cc} = -0.215
\]

Bottom of CIP slab:

\[
\sigma_{4cc} = -(M_{dc} + M_{lx})C_{4cc} x n / lcc
\]

\[
\sigma_{4cc} = -0.168
\]

b) Stresses at mid-span for \((M_\text{o} + M_{dc} + M_{dcc} + M_{lx})\):

Top of PC member:

\[
\sigma_{1cc} = \frac{P_{ee}}{A_c} + \frac{(P_{ee} * e * C_{1c})}{I_c} - (M_\text{o} + M_{dc})C_{1c}/I_c - (M_{dcc} + M_{lx})C_{1cc}/I_c
\]

\[
\sigma_{1cc} = -2.154
\]

Bottom of PC member:

\[
\sigma_{2cc} = \frac{P_{ee}}{A_c} - \frac{(P_{ee} * e * C_{2c})}{I_c} + (M_\text{o} + M_{dc})C_{2c}/I_c + (M_{dcc} + M_{lx})C_{2cc}/I_c
\]

\[
\sigma_{2cc} = 0.745
\]

Top of CIP slab:

\[
\sigma_{3cc} = -(M_{dc} + M_{lx})C_{3cc} x n / lcc
\]

\[
\sigma_{3cc} = -0.363
\]

Bottom of CIP slab:

\[
\sigma_{4cc} = -(M_{dc} + M_{lx})C_{4cc} x n / lcc
\]

\[
\sigma_{4cc} = -0.284
\]

A-40
A.9.4. Supplementary Non-prestressed Reinforcing for Initial Conditions

a) Steel specifications

Bar size from 3 to 6: (i.e. enter 5 for #5 bar)  
Yield stress of reinforcing steel (ksi):  
Area of chosen bar size (in^2):  
ACI recommended allowable stress [ACI R18.4.1 (b) and (c)]:

<table>
<thead>
<tr>
<th>Size</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade</td>
<td>50.0</td>
</tr>
<tr>
<td>Abar</td>
<td>0.31</td>
</tr>
<tr>
<td>fyal</td>
<td>30.00</td>
</tr>
</tbody>
</table>

b) Tensile force to be taken be supplementary reinforcement

1) Tensile forces due to member end stresses resulting from (Moe):

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile member end tensile stress (ksi):</td>
<td>0.355</td>
</tr>
<tr>
<td>Tension at top of PC member at transfer:</td>
<td>0.713</td>
</tr>
<tr>
<td>Comp. at bottom of PC member at transfer:</td>
<td>-1.522</td>
</tr>
</tbody>
</table>

Depth of tension zone (in):

\[ cs = \frac{f_{1le}}{(f_{1le} + f_{2le})} \times h_c \]  
\[ cs = 6.59 \]  

Tension force to be resisted (kips):

\[ T_{fs} = cs \times f_{1le} \times \frac{b_c}{2} \]  
\[ T_{fs} = 51.68 \]

2) Tensile forces due to stresses at point \( x_r \) resulting from (Mox):

* These equations assume that \( x_r \) falls at the location of a gap.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension at top of PC member at transfer:</td>
<td>0.177</td>
</tr>
<tr>
<td>Comp. at bottom of PC member at transfer:</td>
<td>0.439</td>
</tr>
<tr>
<td></td>
<td>-2.094</td>
</tr>
</tbody>
</table>

Depth of tension zone (in):

\[ cg = \frac{f_{1lx}}{(f_{1lx} + f_{2lx})} \times h_c \]  
\[ cg = 3.47 \]  

Tension force to be resisted (kips):

\[ T_{fc} = cg \times f_{1lx} \times \frac{b_c}{2} \]  
\[ T_{fc} = 25.90 \]
3) Tensile forces due to stresses at center span for (Mo):

* These calculations assume that there is a gap at the center

| Allowable mid-span tensile stress (ksi): | fllc | 0.177 |
| Tension at top of PC member at transfer: | fll | 0.186 |
| Comp. at bottom of PC member at transfer: | fll | -1.841 |

Depth of tension zone (in): 

\[ cc = \frac{fll}{(fll + f2)} \times h2c \]  

(abs. values)

| cc | 1.84 |

Tension force to be resisted (kips):

\[ Tfc = cc \times fll \times \frac{bc}{2} \]

| Tfc | 10.98 |

c) Required supplementary reinforcement

* The supplementary reinforcement bar size is defined in Sect. A.6.2.

1) Required reinforcing bars in the top portion of the first stub:

| Required steel area: | Asts | 1.72 |
| Required number of bars | Stubbars | 6 |

2) Required reinforcing bars in the top portion of first gap:

| Required steel area: | Astg | 0.86 |
| Required number of bars | Gapbars | 3 |

3) Required reinforcing bars in the top portion of the center region:

| Required steel area: | Astc | 0.37 |
| Required number of bars | Centerbars | 2 |
A.9.5. Deflection Calculations Using The Bilinear Moment and Factor Method

a) Cracked Section Parameters: [PCI 4.6.3]

Cracked moment of inertia (in^4):

\[ Ep = 28.5 \times 10^6 \text{ psi} \]

\[ nps = Ep / Ec \]

\[ I_{cr} = nps^*Aps^*dp^*/2(1-1.6*sqrt(nps*rp^*)) \]

Portion of the applied loads that lead to the rupture stress of the PC bottom:

Tensile stress under applied loads:

\[ f_{2cc} = 0.745 \]

PC rupture strength 7.5 * sqrt(f'c):

\[ fr = 0.530 \]

Final stress above the ruptures strength:

\[ f_{2cc} - fr = 0.214 \]

Stresses due to dead and live loads:

\[ fl = (MI) * 12 * C2cc / lcc \]

\[ fl_{0} = fl - (Diffl) / ll * WI \]

\[ Wlc = (WI - Wlg) \]

Fraction of (WI) which is taken by lcr & Icc:

\[ Diffl = (f_{2cc} - fr) \]

\[ Diffl = 0 \text{ if } f_{2cc} < 0 \]

\[ Wlg = (fl - (Diffl)) / fl * WI \]

\[ Wlcr = (WI - Wlg) \]

b) Deflection Calculations

\[ Y_{pl} = (PI x e x L^2) / (8 Ec Ic) \]

\[ Y_{pe} = Y_{pl} (Pe/Pl) \]

\[ Y_{o} = 5(Wo x L^4) / (384 Ec Ic) \]

\[ Y_{dc} = 5(Wdc x L^4) / (384 Ec Ic) \]

\[ Y_{dce} = 5(Wdce x L^4) / (384*Ec Icc) \]

\[ Y_{lg} = 5(Wlg x L^4) / (384 Ec Icc) \]

\[ Y_{lcr} = 5(Wlcr x L^4) / (384 Ec Icr) \]
Table of deflection calculations (units = inches):

<table>
<thead>
<tr>
<th>Type of Def</th>
<th>Deflection</th>
<th>Factor</th>
<th>Final Defl.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ypl</td>
<td>0.671</td>
<td>2.20</td>
<td>1.476</td>
</tr>
<tr>
<td>Yo</td>
<td>-0.212</td>
<td>2.40</td>
<td>-0.509</td>
</tr>
<tr>
<td>Ydc</td>
<td>-0.637</td>
<td>2.30</td>
<td>-1.465</td>
</tr>
<tr>
<td>Ydnc</td>
<td>-0.015</td>
<td>3.00</td>
<td>-0.045</td>
</tr>
<tr>
<td>Ylg</td>
<td>-0.011</td>
<td>1.00</td>
<td>-0.011</td>
</tr>
<tr>
<td>Ylcr</td>
<td>-0.305</td>
<td>1.00</td>
<td>-0.305</td>
</tr>
<tr>
<td>Yshort</td>
<td>-0.509</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ylong</td>
<td>-0.859</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

c) Initial deflections:

* Note: Portion of the live load deflection uses cracked section

| Yshort      | -0.509 | in    | Positive upwards |

d) Long-term deflections:

* Note: Portion of the live load deflection uses cracked section

| Ylong       | -0.859 | in    | Positive upwards |
A.10. CALCULATION OF INITIAL PRESTRESS FORCE LIMITS

Radius of gyration: \( kr = 5.77 \)

For the limit calculations for Pi involving initial tensile stress limits, Pi has been predefined by its maximum acceptable value of \( 0.75 \cdot fpu \cdot Ap \). This was done because supplementary reinforcement will be used to accommodate any tensile stresses in the member which cannot be taken by the concrete.

A.10.1. Prestress Force Limits For Initial Load Stage

a) Limits based on member end region (Moe): \( \text{Reduced Strands} \)

\[ P_{1er} \leq \frac{1}{(e_i \cdot l_s / (C1cs^*Acs))} \cdot (ftis \cdot lcs/C1cs + Moe \cdot 12) \]

\[ P_{1er} = 2212.2 \]

* Note: the above equation for \( P_{1er} \) has been redefined as \( P_{1er} = 0.74 \cdot fpu \cdot Ap \)

\[ P_{1er} = 580.8 \]

\[ P_{2er} \leq (-fci + Moe \cdot 12 \cdot C2cs/lcs) \cdot Acs / (1 + ei \cdot C2cs / (lcs / Acs)) \]

\[ P_{2er} = 1172.4 \]

b) Limits based on full strand pattern at point \( x_r \) (Mox):

\[ P_{1x} \leq \frac{1}{(e_i - K2c)} \cdot (ftis \cdot S1c + Mox \cdot 12) \]

69.15 2266.67 503.6

\[ P_{1x} = 503.6 \]

* Note: the above equation for \( P_{1x} \) has been replaced by \( P_{1x} = 0.74 \cdot fpu \cdot Ap \)

\[ P_{1x} = 580.8 \]

\[ P_{2x} \leq (-fci + Mox \cdot 12 / S2c) \cdot Ac / (1 + e \cdot C2c / kr^2) \]

\[ P_{2x} = 563.9 \]

c) Limits based on full strand pattern at midspan for (Mo):

\[ P_{1m} \leq \frac{1}{(e_i - K2c)} \cdot (ftim \cdot S1c + Mo \cdot 12) \]

\[ P_{1m} = 556.4 \]

* Note: the above equation for \( P_{1m} \) has been replaced by \( P_{1m} = 0.74 \cdot fpu \cdot Ap \)

\[ P_{1m} = 580.8 \]

\[ P_{2m} \leq (-fci + Mo*12 / S2c)Ac / (1 + e \cdot C2c / kr^2) \]

\[ P_{2m} = 621.8 \]
A.10.2. Prestress Force Limits For Intermediate Non-Composite Load Stage

a) Limits based on point \( x_r \) (\( M_{ox} + M_{dcx} \)): (Reduced Strands)

\[
P_{1xri} \geq (fcs \cdot S_{1c} + (M_{ox} + M_{dcx})) \cdot 1/(1-R) \cdot 1/(9 - k_2c)
\]

\[
P_{1xri} = -644.2
\]

\[
P_{2xri} \geq (-fts \cdot S_{2c} + (M_{ox} + M_{dcx})) \cdot 1/(1-R) \cdot 1/(e + k_1c)
\]

\[
P_{2xri} = 166.1
\]

b) Limits based on full strand pattern at midspan (\( M_{o} + M_{dc} \)):

\[
P_{1si} \geq (fcs \cdot S_{1c} + (M_{o} + M_{dc})) \cdot 1/(1-R) \cdot 1/(e - k_2c)
\]

\[
P_{1si} = 188.5
\]

\[
P_{2si} \geq (-fts \cdot S_{2c} + (M_{o} + M_{dc})) \cdot 1/(1-R) \cdot 1/(e + k_1c)
\]

\[
P_{2si} = 438.7
\]

A.10.3. Prestress Force Limits For Full Composite Load Stage

a) Limits based on point \( x_r \) (\( M_{ox} + M_{dcx} + M_{dccc} + M_{lx} \)): (Reduced Strands)

\[
P_{1xr} \geq (fcs \cdot S_{1c} + (M_{ox} + M_{dcx}) + (S_{1c} \cdot S_{1cc} \cdot (M_{dccc} + M_{lx}))) \cdot 1/(1-R) \cdot 1/(e - k_2c)
\]

\[
P_{1xr} = -469.9
\]

\[
P_{2xr} \geq (-fts \cdot S_{2c} + (M_{ox} + M_{dcx}) + (S_{2c} \cdot S_{2cc} \cdot (M_{dccc} + M_{lx}))) \cdot 1/(1-R) \cdot 1/(e + k_1c)
\]

\[
P_{2xr} = 222.7
\]

b) Limits based on full strand pattern at midspan (\( M_{o} + M_{dc} + M_{dccc} + M_{l} \)):

\[
P_{1s} \geq (fcs \cdot S_{1c} + (M_{o} + M_{dc}) + (S_{1c} \cdot S_{1cc} \cdot (M_{dccc} + M_{l}))) \cdot 1/(1-R) \cdot 1/(e - k_2c)
\]

\[
P_{1s} = 483.3
\]

\[
P_{2s} \geq (-fts \cdot S_{2c} + (M_{o} + M_{dc}) + (S_{2c} \cdot S_{2cc} \cdot (M_{dccc} + M_{l}))) \cdot 1/(1-R) \cdot 1/(e + k_1c)
\]

\[
P_{2s} = 534.5
\]

A.10.4. Summary of Allowable Range For Prestress Force

\[P_{min} = \text{Largest of } (P_{1xri}, P_{2xri}, P_{1si}, P_{2si}, P_{1xr}, P_{2xr}, P_{1s}, P_{2s})\]

\[
P_{min} = 534.52
\]

\[P_{max} = \text{Smallest of } (P_{1er}, P_{2er}, P_{1x}, P_{2x}, P_{1m}, P_{2m})\]

\[
P_{max} = 563.93
\]
### A.11. SHEAR REINFORCEMENT DESIGN FOR THE STUB REGIONS

**Member:** 
Stub Girder 34 ST 38-18-20

#### A.11.1. Initial Data

**a) Design loads:**

<table>
<thead>
<tr>
<th>Member section of interest (ft):</th>
<th>x</th>
<th>0.83 (h/c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear due to self weight at point x (k):</td>
<td>Vox</td>
<td>14.50 unfactored</td>
</tr>
<tr>
<td>Superimposed DL &amp; LL shear at x (k):</td>
<td>Vx</td>
<td>125.12 factored</td>
</tr>
<tr>
<td>Superimposed DL &amp; LL moment at x (k-ft)</td>
<td>Mx</td>
<td>107.28 factored</td>
</tr>
<tr>
<td>Total factored shear at point x (k):</td>
<td>Vux</td>
<td>145.42</td>
</tr>
</tbody>
</table>

**b) Concrete specifications:**

<table>
<thead>
<tr>
<th>Shear reduction factor [ACI 9.3.2.3]:</th>
<th>Ov</th>
<th>0.85</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coefficient for concrete/concrete [ACI 11.7.4]:</td>
<td>uc</td>
<td>1.00</td>
</tr>
<tr>
<td>Width of shear transfer interface at stubs (in):</td>
<td>bvs</td>
<td>22.00</td>
</tr>
<tr>
<td>Effective web width at stubs (in):</td>
<td>bws</td>
<td>22.00</td>
</tr>
<tr>
<td>Effective depth to steel centroid at stubs (in):</td>
<td>dst</td>
<td>38.58</td>
</tr>
<tr>
<td>Total length of shear transfer surface (in):</td>
<td>Ly</td>
<td>264.00</td>
</tr>
<tr>
<td>Number stubs (assuming equal length):</td>
<td>Nstub</td>
<td>4</td>
</tr>
<tr>
<td>Area of shear transfer for one stub (in²):</td>
<td>Acv</td>
<td>1452</td>
</tr>
</tbody>
</table>

**c) Stirrup specifications:**

<table>
<thead>
<tr>
<th>Stirrup bar size (#): (#3 thru #6 only)</th>
<th>#st</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel grade of stirrup (fyr = 60 or 40 ksi):</td>
<td>fyr</td>
<td>50</td>
</tr>
<tr>
<td>Total steel area for one &quot;U&quot; stirrup (in²):</td>
<td>Asr</td>
<td>0.620</td>
</tr>
<tr>
<td>Check for minimum steel requirement</td>
<td>#5 Bar</td>
<td>OK</td>
</tr>
</tbody>
</table>

Smax = smaller of 24"; 3/4" h,  
or fyr * Asr / (50 * b) [ACI 11.5.4.1]  
Smax | 24.00 |
A.11.2. Concrete Shear Strength Calculations [ACI 11.4]

a) Calculation of critical flexural cracking moment at stubs (Mcr):

\[
M_{cr} = \frac{I_{cs}}{C_{2cs}}(6 \times (f_c)^{1/2}) + \frac{I_{cs}}{C_{2cs}}(f_{2p} - f_o)
\]

\[
f_{2p} = \text{flexural stresses due to } P_e \text{ at member bottom (abs. value)}
\]

\[
\frac{P_e}{A_{c}} + P_e \times C_{2c/lc}
\]

\[
f_{2p} = 2.09 \text{ ksi}
\]

\[
f_o = \text{flexural stresses due to unfactored self weight:}
\]

\[
W_o (x/2) (L - x) C_{2c/lc}
\]

\[
\text{fo} = 0.066 \text{ ksi}
\]

\[
M_{cr} = 760.47 \text{ k-ft}
\]

b) Nominal concrete strength at the location of the stubs:

\[
V_{1ci} = 0.6 \times (f_c)^{1/2} \times b_{ws} \times d_{st} + V_{ox} + \frac{(M_{cr}/M_x)}{V_x}
\]

\[
V_{1ci} = 937.44 \text{ k}
\]

\[
V_{2ci} = 1.7 \times (f_c)^{1/2} \times b_{ws} \times d_{st}
\]

\[
V_{2ci} = 102.03 \text{ k}
\]

\[
V_{3ci} < 5 \times (f_c)^{1/2} \times b_{ws} \times d_{st}
\]

\[
V_{3ci} = 300.07 \text{ k}
\]

\[
V_{ci} = \text{greater of } V_{1ci} \text{ and } V_{2ci}, \text{ but not greater than } V_{3ci}
\]

\[
V_{ci} = 300.07 \text{ k}
\]

\[
V_{cw} = (3.5 \times (f_c)^{1/2} + 0.3 \frac{P_e}{A_{1c}}) \times b_{ws} \times d_{st}
\]

\[
V_{cw} = 323.19 \text{ k}
\]

\[
V_{c} = \text{smaller of } V_{ci} \text{ or } V_{cw} \text{ therefore:}
\]

\[
V_{c} = 300.07 \text{ k}
\]
A.11.3. Vertical Shear Reinforcement Calculations [ACI 11.5]

a) Required stirrup spacing given Asr:

\[ Sr = \left( \frac{Ov \cdot Asr \cdot Fyr \cdot dst}{(Vux - Oc \cdot Vc)} \right) \]

If \( Sr \) is negative then \( Sr = S_{max} \):

\[ Sr = 24.00 \text{ in} \]

Therefore try:

\[ S_{trial} = 24.00 \text{ in} \]

b) Check for minimum required shear steel given \( S \)

\[ Av_{min(a)} = \left( \frac{50 \cdot bws \cdot S}{f_{yu}} \right) \]

\[ Av_{min(b)} = \frac{Ap}{80 \cdot f_{pu} \cdot f_{yu} \cdot S \cdot hc \cdot (dst \cdot bws)^{1/2}} \]

\[ Av_{min(a)} = 0.528 \text{ in}^2 \]

\[ Av_{min(b)} = 0.16 \text{ in}^2 \]

\#5 Bar OK \( \text{in}^2 \)

c) Check of limits on shear steel strength:

\[ Vs = \text{Smaller of } Asr \cdot f_{yu} \cdot dst / S \text{ or } 8 \cdot (f_c)^{1/2} \cdot bws \cdot dst \]

\[ Vs = 49.83 \]

If \( Vs > 4 \cdot (f_c)^{1/2} \cdot bws \cdot dst \) then \( S \) must be reduced by \( 1/2 \)

240.06 OK

d) Check of nominal shear strength

\[ V_n = V_{c} + Vs \]

\[ V_n = 349.90 \]

\[ V_{ux} = 145.42 \]

\[ Ov \cdot V_n \geq V_{ux} \]

Ov Vn OK

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A.11.4. **Horizontal Shear Design Calculations for Composite Action**

a) **Calculation of the factored horizontal shear force for end stub:**

\[ xc = \text{point of calculation for design compressive force} = \text{Length of one stub} + \text{length of one gap} \ (\text{ft}) \]

\[ xc = 8.33 \ (\text{ft}) \]

\[ Mc = \text{factored moment resulting from } (W_o + W_{dc} + W_{dcc} + W_l) \text{ at } xc \]

\[ Mc = 931.65 \]

\[ Vu = C = \frac{Mc}{(dst - a/2)} \]

\[ Vu = 307.01 \ (k) \]

b) **Shear strength method** [ACI 17.5]

*Required stirrup spacing by shear strength method*

\[ Vu \leq O_v (260 + 0.6 \cdot rv \cdot f_{yr}) \cdot A_{cv} \]

where: \( rv = \frac{A_{sr}}{(bws \cdot S)} \)

Therefore: \( S = 0.6 \cdot f_{yr} \cdot \frac{A_{sr}}{bws \cdot (1 / (Vu / O_v A_{cv} - 260))} \)

\[ S \leq S_{\text{max}} \]

\[ S = 24.0 \ (\text{in}) \]

*Nominal shear strength of shear interface based on S*

\[ V_{nh} = (260 + 0.6 \cdot rv \cdot f_{yr}) \cdot A_{cv} = 428.67 \ (k) \]

\[ V_{nh} = 428.67 \ (k) \]

*Note: If Vu is greater than \( O_v \cdot 500 \cdot A_{cv} \) then shear friction method must be used.

\[ O_v \cdot 500 \cdot A_{cv} = 617.10 \]

Shear strength method is valid

*Check of nominal shear strength:*

\[ O_v V_{nh} \text{ - OK} \]

Shear Strength - OK

A-50
c) Shear friction method \[ \text{[ACI 11.7]} \]

**Required total stirrup area for one stub:**

\[
A_{fricr} = \frac{V_u}{(O_v \cdot u_c \cdot f_{yr})} \quad A_{fricr} = 7.22 \text{ in}^2
\]

**Required number of stirrups for one stub:**

\[
N_{fricr} = \left( \frac{A_{fricr}}{A_{sr}} \right) \quad N_{fricr} = 12
\]

**Required stirrup spacing across one stub for shear friction method:**

\[
S_{fricr} = \frac{(L_v / N_{stub}) / (N_{fricr} - 1)}{L_v/N_{stub} = \text{stub length}} \quad S_{fricr} = 5.90
\]

**Check of shear friction strength limits:**

\[
V_n < \text{smaller of } 0.2 \cdot f_{cc} \cdot A_{cv} \text{ or } 800 \cdot A_{cv}
\]

\[
V_n = A_{sr} \cdot N_{fric} \cdot u_c \cdot f_{yr} = 372.00 \text{ k}
\]

\[
0.2 f_{cc} A_{cv} = 1016.40 \text{ k}
\]

\[
800 A_{cv} = 1161.60 \text{ k}
\]

\[
V_n \text{ OK}
\]

**A.11.5. Summary of Shear Reinforcement Requirements For the Stubs**

**Required spacing:**

\[
S_{regs} = \text{smallest of that required for vertical shear, minimum steel, or composite action.}
\]

- **Spacing for minimum steel:**
  \[
  S_{max} = 24.00 \text{ in}
  \]

- **Spacing for vertical shear:**
  \[
  S_{vert} = 24.00 \text{ in}
  \]

- **Spacing for horizontal shear:**
  \[
  S_{horiz} = 24.00 \text{ in}
  \]

- **Controlling spacing for stub region:**
  \[
  S_{regs} = 24.00 \text{ in}
  \]
A.12. SHEAR REINFORCEMENT DESIGN FOR THE GAP REGIONS

Member:  
Stub Girder 34 ST 38-18-20

*Note: calculations assumes shear is taken by the continuous lower flange

A.12.1. Initial Data

a) Design loads:

Member section of interest (ft):  
xg 5.5 stub / 12

Shear due to self weight at point x (k):  
Voxg 9.81 unfactored

Superimposed DL & LL shear at x (k):  
Vxg 84.62 factored

Superimposed DL & LL moment at x (k-ft):  
Mxg 596.67 factored

Total factored shear at point x (k):  
Vuxg 98.35

b) Concrete specifications:

Width of shear transfer interface at gaps:  
bvg 34.00 in

Effective web width at gaps:  
bwe 34.00 in

Effective depth to steel centroid at gaps:  
dg 16.58 in

c) Stirrup specifications: (these values set by stub shear specs.)

Stirrup bar size (#):  
#st 5

Steel grade of stirrup (fyr = 60 or 40 ksi):  
fyr 60

Total steel area for one "U" stirrup (in^2):  
Asr 0.620

Check for minimum steel requirement

Smax = smaller of 24" ; 3/4 * h2c,  
or fyr * Asr / (50 * bw [ACI 11.5.4.1])  
Smaxg 15.00

[Image]
A.12.2. Concrete Shear Strength Calculations

a) Calculation of critical flexural cracking moment at gaps (Mcr):

\[ M_{cr} = \frac{l_c}{C_{2c}} * (6 * (f_{c})^{1/2}) + l_c / C_{2c} * (f_{2p} - f_{og}) \]

- \( f_{2p} \) = flexural stresses due to \( P_e \) at member bottom (absv)
  \[ f_{2p} = \frac{P_e/A_c + P_e * \varepsilon_c C_{2c}/l_c}{f_{og}} \]
  \( f_{2p} \) = 2.091 ksi

- \( f_{og} \) = flexural stresses due to unfactored self weight:
  \[ f_{og} = \omega (x/2) (L - x) C_{2c}/l_c \]
  \( f_{og} \) = 0.366 ksi

\( M_{cr} = 703.75 \text{ k-ft} \)

b) Nominal Concrete Strength at the Location of the gaps:

- \( V_{1cig} = 0.6 * (f_{c})^{1/2} * bwg * dg + V_{og} + (M_{cr}/M_{xg}) * V_{xg} \)
  \( V_{1cig} = 133.53 \text{ k} \)

- \( V_{2cig} = 1.7 * (f_{c})^{1/2} * bwg * dg \)
  \( V_{2cig} = 67.76 \text{ k} \)

- \( V_{3cig} = 5 * (f_{c})^{1/2} * bwg * dg \)
  \( V_{3cig} = 199.29 \text{ k} \)

\( V_{cig} = \) greater of \( V_{1cig} \) and \( V_{2cig} \), but not greater than \( V_{3cig} \)
  \( V_{cig} = 133.53 \text{ k} \)

- \( V_{cwg} = (3.5 * (f_{c})^{1/2} + 0.3 P_e/A_c) * bwg * dg \)
  \( V_{cwg} = 258.40 \text{ k} \)

\( V_{cg} = \) smaller of \( V_{cig} \) or \( V_{cwg} \) therefore:
  \( V_{cg} = 133.53 \text{ k} \)
A.12.3. Vertical Shear Reinforcement Calculations [ACI 11.5]

a) Required stirrup spacing given Assr:
\[ Sr = \frac{(Ov \cdot Assr \cdot Fyr \cdot dg)}{(Vuxg - Oc \cdot Vcg)} \]
If \( Sr \) is negative then \( Sr = S_{max} \):
Therefore try:
\[ S_{rig} \]
\[ S_{trial} \]

b) Check for minimum required shear steel given \( S \)
\[ Avmax = \frac{(50 \cdot bwg \cdot S)}{fyr} \]
\[ Avminb = \frac{Ap}{80 \cdot fpu / fyr \cdot S / h2c \cdot (dg / bwg)^{1/2}} \]

Avminag = 0.510 in^2
Avminbg = 0.10 in^2
#5 Bar OK in^2

c) Check of limits on shear steel strength:
\[ Vsg = \text{Smaller of } Assr \cdot fyr \cdot dg \leq S \text{ or } 8 \cdot (f’c)^{1/2} \cdot dg \cdot bwg \]
If \( Vsg > 4 \cdot (f’c)^{1/2} \cdot bwg \cdot dg \) then \( S \) must be reduced by 1/2

159.43 OK

d) Check of nominal shear strength
\[ Vng = Vcg + Vsg = \]
\[ Vng \]
167.79
Ov Vng = 142.62
Vuxg = 98.35
Ov Vng = Vuxg = OK

A.12.4 Summary of Shear Reinforcement Requirements For the Stubs

Required spacing:
\[ S_{regs} = \text{smallest of that required for vertical shear, minimum steel, or composite action.} \]

Spacing for minimum steel:
\[ S_{max} \]
15.00 in

Spacing for vertical shear:
\[ S_{vert} \]
15.00 in

Controlling spacing for stub region:
\[ S_{regg} \]
15.00 in
A.13 TORSION REINFORCEMENT DESIGN

Member: Stub Girder 34 ST 38-18-20

A.13.1 Design Loads

a) Factored loads causing torsion at critical section

Critical section (from left end) (in): xt 68.00

Constructions live loads contributing to torsion: Qcon 25 psf

Pst = \frac{Atr}{2} \times (1.7 \times Qcon + 1.4 \times Qo) / (Nfloor \times 4)

\begin{align*}
Pst = & 2.631 \text{k} \\
\text{where: } & Qo = \text{dead weight of members framing into girder (psf)} \\
& Nfloor = \text{number of floor member framing into one side of girder} \\
& Atr/2 = \text{half the total tributary area (only one side is loaded)}
\end{align*}

Uniform load approximation of torsional loading

\begin{align*}
Wtor = & (Nfloor \times 4) \times (Pst) / L \\
& \frac{1.4 \times Qo + Wtor}{U2} \times \frac{xt}{12}
\end{align*}

Shear force causing torque at critical point

\begin{align*}
Vtor = & 13.46 \text{k} \\
& 1.38 \text{klf}
\end{align*}

b) Total factored shear load at xt acting simultaneously with Vtor

\begin{align*}
Vut = & (1.4 \times Qo + Wtor) / (L/2 - xt/12) \\
& 27.19 \text{k}
\end{align*}

c) Torsional loading at critical point xt

Eccentricity through which Vtor acts causing torsional moment Tu

et = \frac{bc}{2} - \frac{1}{4} \times b2c \\
et = 15.50 \text{ in}

Torsional moment Tu (k-in)

\begin{align*}
Tu = & 208.66 \text{k-in}
\end{align*}

A.13.2 Check of Nominal Concrete Torsion Strength [ACI 11.6.6]

a) Geometric parameters of critical section

\begin{align*}
\text{Long side of section rectangle (in): } & ys \quad 34.00 \\
\text{Short side of section rectangle (in): } & xs \quad 20.00
\end{align*}
b) Nominal concrete torsion strength

\[ T_{nc} = O_v \cdot Y_t \cdot (1.5 \cdot B_{tor} \cdot \text{sum}(x^2 \cdot y) \cdot a \cdot \sqrt{f_{cc}}) \]

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( Y_t )</td>
<td>1.55</td>
</tr>
<tr>
<td>( B_{tor} )</td>
<td>0.15</td>
</tr>
</tbody>
</table>

\[ Y_t = \sqrt{1 + 10 \cdot \frac{Pe}{A_{cg}} \cdot \frac{1}{f_{cc}}} \]

\[ B_{tor} = 0.35 \left( 0.75 + \frac{x}{y} \right) \]

\( a = \text{function of concrete density} \)
\( a = 1.00 \) (normal weight concrete)
\( f_{cc} = \frac{Pe}{A_{c}} \)
\( O_v = \text{reduction factor} = 0.85 \)

\[ T_{nc} = 293.68 \text{ k-in} \]

c) Check of minimum to actual torsion moment

If \( T_u \leq T_{umin} \) then torsion may be disregarded for design

\( T_u \leq T_{nc} \) \hspace{1cm} \text{OK}

\( \text{Torsion may be disregarded} \)
A.14. **STUB GIRDER DESIGN SUMMARY - Unshored Construction**

**Member:** Stub Girder 34 ST 38-18-20

### A.14.1. Design Parameters

**a) Design layout**
- **Bay Dimensions:**
  - 32.00 ft x 40.00 ft
- **Girder length:**
  - L 30.50 ft
- **Double tee floor member:**
  - 8' - 20" DT

**b) Service loads**

<table>
<thead>
<tr>
<th>Loads</th>
<th>(plf)</th>
<th>Moments</th>
<th>(k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wo</td>
<td>1005.9</td>
<td>Mo</td>
<td>117.0</td>
</tr>
<tr>
<td>Wdc</td>
<td>3021.6</td>
<td>Mdc</td>
<td>351.4</td>
</tr>
<tr>
<td>Wdcc</td>
<td>629.5</td>
<td>Mdcc</td>
<td>73.2</td>
</tr>
<tr>
<td>WI</td>
<td>2098.4</td>
<td>MI</td>
<td>244.0</td>
</tr>
<tr>
<td></td>
<td>6755.4</td>
<td></td>
<td>785.5</td>
</tr>
</tbody>
</table>

### A.14.2. Member Dimensions:

Space between stubs - gap (in):
- Gap 34.00
- Ngap 3
- Nstub 4

Effective compression slab thickness (in):
- $t$ 4.00
- $te$ 5.00

Average member weight / ft (plf):
- $Wt$ 1.006
- Total Wt 30,679 Ib
A.14.3. **Material Specifications**

Precast concrete (psi):
Green precast concrete (psi): (0.7 to 0.75)
CIP (psi):
Modular ratio (Eci/Ec):

<table>
<thead>
<tr>
<th>Ultimate strength of prestressing steel (ksi):</th>
</tr>
</thead>
<tbody>
<tr>
<td>fpu 270</td>
</tr>
</tbody>
</table>

PC allowable tensile stress coefficient Kc *sqrt( fc)

| Kc 12                                      |

A.14.4. **Prestressing Steel Specifications**

a) **Strand Placement**

(3" lateral cover provided for all strands)

<table>
<thead>
<tr>
<th>Allowable number of strands per row with 2&quot; spacing:</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Row From Bottle</th>
<th>Distance From Bottm.</th>
<th>Distance from Precast NA</th>
<th>Number of Strands /Row</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row #1</td>
<td>3.00</td>
<td>7.00</td>
<td>15</td>
</tr>
<tr>
<td>Row #2</td>
<td>5.00</td>
<td>5.00</td>
<td>4</td>
</tr>
<tr>
<td>Row #3</td>
<td>7.00</td>
<td>3.00</td>
<td></td>
</tr>
<tr>
<td>Row #4</td>
<td>9.00</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tendon Eccentricity (in):</th>
</tr>
</thead>
<tbody>
<tr>
<td>e 6.58</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Eccentricity for composite member:</th>
</tr>
</thead>
<tbody>
<tr>
<td>ecc 15.50</td>
</tr>
</tbody>
</table>

b) **Bond release specifications:**

<table>
<thead>
<tr>
<th>No. of tendons furthest from the NA to be released:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nr 0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Release Length for all bond released tendons (in):</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rel. Length 0.00</td>
</tr>
</tbody>
</table>

c) **Prestressing specifications:**

Strand Type:

| Low - relaxation 7-Wire Strand |
|---------------------------------

<table>
<thead>
<tr>
<th>Number of strands:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ns 19</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Strand diameter (in):</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ds 0.50</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Total area of prestress steel (in^2):</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ap 2.907</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Prestressing steel ultimate strength (ksi):</th>
</tr>
</thead>
<tbody>
<tr>
<td>fpu 270</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Prestress losses:</th>
</tr>
</thead>
<tbody>
<tr>
<td>R 0.15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial prestress force (k):</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pl 562.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial prestress (ksi):</th>
</tr>
</thead>
<tbody>
<tr>
<td>fpl 193</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Effective prestress force (k):</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pe 478.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Effective prestress (ksi):</th>
</tr>
</thead>
<tbody>
<tr>
<td>fpe 164.5</td>
</tr>
</tbody>
</table>
A.14.5. **Required Supplementary Top Reinforcement For Initial Conditions**

**a)** Supplementary reinforcement specifications:
- Grade of supplementary reinforcement: 50
- Size of supplementary reinforcing bars: # 5 Bars

**b)** Supplementary reinforcement requirements
- Reinforcing required at top of first stub: 6 Bars
- Reinforcing required at top of first gap: 3 Bars
- Reinforcing required at top of center PC region: 2 Bars
  
  * assumes a gap at center region

A.14.6. **Shear and Torsion Reinforcement**

**a)** Size and grade of stirrups used for shear reinforcement:
- Size of stirrups: #5 Bar
- Grade of stirrups: 50

**b)** Required stirrup spacing for stubs:
  
  If spacing is based on shear friction = 5.90
  
  Sstub = 24.00

**c)** Required stirrup spacing at the locations of the gaps:
  
  Sgap = 15.00

**d)** Torsion:
  
  Torsion may be disregarded

A.14.7. **Flexural Characteristics**

**a)** Nominal strength:
- Non-Composite moment capacity (k-ft): ObMni = 750.60
- Composite moment capacity (k-ft): ObMn = 2074.10

**b)** Estimated deflections:
- Estimated short-term deflections (in): -0.509
- Estimated long-term deflections (in): -0.859
- Estimated deflections affecting non-structural elements (in): -0.681
  
  * Deflections based on upward camber being positive:

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A.15. **NOTATION**

A1 = Area of non-composite stub girder cross-section at the location of the stubs.
A2 = Area of non-composite stub girder cross-section at the location of the gaps.
A3 = Area of composite stub girder cross-section at the location of the gaps.
A4 = Area of composite stub girder cross-section at the location of the stubs.
Acs = Area of non-composite stub girder cross-section at the location of the gaps.
Accs = Area of composite stub girder cross-section at the location of the gaps.
Aesc = Area of non-composite stub girder cross-section at the location of the stubs.
Aces = Area of composite stub girder cross-section at the location of the stubs.
Acg = Area of non-composite stub girder cross-section at the location of the gaps.
Aecg = Area of composite stub girder cross-section at the location of the gaps.
Ac = Stub girder cross-sectional area used for non-composite flexural analysis.
Acc = Stub girder cross-sectional area used for composite flexural analysis.
Ap = Total area of prestressing steel.
Ape = Area of effective prestressing steel at the ends of the stub girder.
Acv = Surface area of the top of one stub.
Asr = Total area of one hoop stirrup ( = 2 x the area of the bar).
Atr = Tributary area for the stub girder.
Amin = Minimum area of shear reinforcement steel required by the ACI Code.
Afri = Total steel area required in one stub for horizontal shear transfer by shear friction method.
Abar = Area of one of the reinforcing bars used for supplementary top steel.
Ast = Required area of supplementary top reinforcement at the first stub.
Astg = Required area of supplementary top reinforcement at the first gap.
Astc = Required area of supplementary top reinforcement at mid-span.
ai = Depth of the effective compression block at the non-composite ultimate load (calculated at a gap region).
aic = Depth of the effective compression block in the cast-in-place compression flange at the composite ultimate load.
Bi = Beta factor for the precast concrete which translates the depth of the actual compression zone to an equivalent depth with a uniform stress distribution (the beta factor is a function of the concrete strength [ACI 10.7.2.3]).
B = Beta factor for the cast-in-place concrete which translates the depth of the actual compression zone to an equivalent depth with a uniform stress distribution (the beta factor is a function of the concrete strength [ACI 10.7.2.3]).
Btor = Variable which holds the value of a portion of the calculation carried out to evaluate the nominal torsion strength of the chosen concrete section (Btor = 0.35(0.75 + xs / ys)).
bc = Total width of the base of the stub girder.
b1c = Width of the stub of the stub girder.
b2c = Width of the bearing surface of the lower flange of the stub girder.
btrc = Effective transposed width of the cast-in-place compression flange.
btr = Effective transposed width of the cast-in-place compression flange.
bwg = Effective web width at the location of the gaps.
bws = Effective web width at the location of the stubs.
bv = Width of the shear transfer width at the location of the stubs ( = width of stubs).
Centerbars = Number of supplementary reinforcing bars needed at the top of the girder at mid-span.
Cl1c = Distance from the top of the stubs to the non-composite centroidal axis calculated for the stub regions.
Cl2c = Distance from the top of the lower flange to the non-composite centroidal axis calculated for the gap regions.
C33c = Distance from the top of the cast-in-place slab to the composite centroidal axis calculated for the gap regions
C34c = Distance from the top of the cast-in-place slab to the composite centroidal axis calculated for the stub regions.
C1cs = Distance from the top of the stubs to the non-composite centroidal axis calculated for the stub regions.
C2cs = Distance from the bottom of the stub girder to the non-composite centroidal axis calculated for the stub regions.
C1ccs = Distance from the top of the stubs to the composite centroidal axis calculated for the stub regions.
C2ccs = Distance from the bottom of the stub girder to the composite centroidal axis calculated for the stub regions.
C3s = Distance from the top of the cast-in-place slab to the composite centroidal axis calculated for the stub regions.
C4s = Distance from the bottom of the cast-in-place slab to the composite centroidal axis calculated for the stub regions.
C1cg = Distance from the top of the lower flange to the non-composite centroidal axis calculated for the gap regions.
C2cg = Distance from the bottom of the stub girder to the non-composite centroidal axis calculated for the gap regions.
C1ccg = Distance from the top of the lower flange to the composite centroidal axis calculated for the gap regions.
C2ccg = Distance from the bottom of the stub girder to the composite centroidal axis calculated for the gap regions.
C3g = Distance from the top of the cast-in-place slab to the composite centroidal axis calculated for the gap regions.
C4g = Distance from the bottom of the cast-in-place slab to the composite centroidal axis calculated for the gap regions.
C1c = Distance from the top of the lower flange to the neutral axis used for the non-composite flexural analysis (i.e. that of the non-comp. member w/o stubs).
C2c = Distance from the bottom of the girder to the neutral axis used for the non-composite flexural analysis (i.e. that of the non-comp. member w/o stubs).
C1cc = Distance from the top of the lower flange to the neutral axis used for the composite flexural analysis (i.e. that of composite member w/o the stubs).
C2cc = Distance from the bottom of the stub girder to the composite centroidal axis used for flexural analysis (i.e. that of the composite member w/o the stubs).
C3cc = Distance from the top of the cast-in-place slab to the composite centroidal axis used for flexural analysis (i.e. that of the composite member w/o stubs).
C4cc = Distance from the bottom of the cast-in-place slab to the composite centroidal axis used for flexural analysis (i.e. that of the composite member w/o stubs).
es = Depth of the tension zone at the end of the member for the initial loading stage.
CG = Depth of the tension zone at the beginning of the first gap for the initial loading stage.
CC = Depth of the tension zone at the centerline the initial loading stage.
Ds = Diameter of prestressing strands.
dg = Depth from outer most compression fiber to the steel centroid for the gap regions.
dst = Depth from outer most compression fiber to the steel centroid for the stub regions.
dp = Depth from the outer most compression fiber to the steel centroid for the composite member.
\[ Ec \] = Modulus of elasticity for the precast concrete.
\[ Ec_{ip} \] = Modulus of elasticity for the cast-in-place concrete.
\[ Ep \] = Modulus of elasticity for the prestressing steel.
\[ et \] = Eccentricity through which the load acts which causes torsion about the centerline of the girder.
\[ e \] = Prestressing steel eccentricity for the non-composite member.
\[ ecc \] = Prestressing steel eccentricity for the composite member.
\[ ecs \] = Prestressing steel eccentricity at the stubs for the non-composite member.
\[ eccs \] = Prestressing steel eccentricity at the stubs for the composite member.
\[ ecg \] = Prestressing steel eccentricity at the gaps for the non-composite member.
\[ eccg \] = Prestressing steel eccentricity at the gaps for the non-composite member.
\[ eics \] = Effective prestressing steel eccentricity at the non-composite member ends, accounting for bond released tendons, if they exist.
\[ f'c \] = Precast concrete strength.
\[ f'ci \] = Green precast concrete strength.
\[ f'cc \] = Cast-in-place concrete strength.
\[ fyr \] = Yield stress of stirrups.
\[ fyal \] = ACI Code recommended yield stress for supplementary top steel.
\[ fpu \] = Tensile strength of the prestressing steel.
\[ fpi \] = Initial prestress immediately after prestress transfer.
\[ fpe \] = Effective prestress after all losses.
\[ fpsi \] = Estimated tendon stress at failure for the non-composite stage of loading.
\[ fps \] = Estimated tendon stress at failure for the composite stage of loading.
\[ fr \] = Rupture strength of the concrete.
\[ fci \] = Allowable initial concrete compressive stress \((6 \times \sqrt{f'c})\).
\[ ftic \] = Allowable initial concrete tensile stress at mid-span \((3 \times \sqrt{f'c})\).
\[ ftis \] = Allowable initial concrete tensile stress at member ends \((6 \times \sqrt{f'c})\).
\[ fcs \] = Allowable concrete compressive stress at service loads \((0.45 \times f'c)\).
\[ fts \] = Allowable concrete tensile stress at member ends \((12 \times \sqrt{f'c})\).
\[ fcms \] = Allowable compressive stress in the cast-in-place slab \((0.45 \times \sqrt{f'c})\).
\[ ftss \] = Allowable tensile stress in the cast-in-place slab \((12 \times \sqrt{f'c})\).
\[ fl \] = Maximum bottom tensile stress due to live load only.
\[ f2p \] = Absolute value of the bottom compressive stress due to the prestress force.
\[ fo \] = Bottom tensile stress due to the self weight of the stub girder at point \(x\).
\[ f2pg \] = Absolute value of the bottom compressive stress due to the prestress force.
\[ fog \] = Bottom tensile stress due to the self weight of the stub girder at point \(xg\).
\[ fli \] = Top concrete stress at mid-span for initial loading.
\[ f2i \] = Bottom concrete stress at mid-span for initial loading.
\[ f1ie \] = Top concrete stress at member ends for initial loading - will reflect the reduced number of effective strands, should bond releasing be used.
\[ f2ie \] = Bottom concrete stress at member ends for initial loading - will reflect the reduced number of effective strands, should bond releasing be used.
\[ f1ix \] = Top concrete stress at point \(xr\) - uses full strand pattern.
\[ f2ix \] = Bottom concrete stress at point \(xr\) - uses full strand pattern.
\[ f1ixr \] = Top concrete stress at point \(xr\) for non-composite load stage - will reflect the reduced number of effective strands, should bond releasing be used.
\[ f2ixr \] = Bottom concrete stress at point \(xr\) for non-composite load stage - will reflect the reduced number of effective strands, should bond releasing be used.
\[ f1s \] = Top concrete stress at mid-span for non-composite load stage.
\[ f2s \] = Bottom concrete stress at mid-span for non-composite load stage.
\[ f1ccx \] = Top concrete stress at point \(x\) for the full service load - will reflect the reduced number of effective strands, should bond releasing be used.
Bottom concrete stress at point xr for the full service load - will reflect the reduced number of effective strands, should bond releasing be used.

Stress at the top of the cast-in-place slab at point xr - will reflect the reduced number of effective strands, should bond releasing be used.

Stress at the bottom of the cast-in-place slab at point xr - will reflect the reduced number of effective strands, should bond releasing be used.

Top concrete stress at mid-span for the full service load.

Bottom concrete stress at mid-span for the full service load.

Stress at the top of the cast-in-place slab at mid-span for the full service load.

Stress at the bottom of the cast-in-place slab at mid-span for the full service load.

Length of the gaps between stubs of the stub girder.

Number of supplementary top reinforcing bars required at the first gap.

Total depth of the non-composite stub girder.

Total depth of the non-composite stub girder.

Total depth of the composite stub girder.

Height of the stubs of the stub girder.

Height of the lower flange of the stub girder.

Moment of inertia of the non-composite stub girder at the stubs.

Moment of inertia of the non-composite stub girder at the gaps.

Moment of inertia of the composite stub girder at the stubs.

Moment of inertia of the composite stub girder at the gaps.

Moment of inertia of the composite stub girder at the stubs.

Height of the lower flange of the stub girder.

Non-composite moment of inertia used for flexural analysis.

Composite moment of inertia used for flexural analysis.

Estimated cracked moment of inertia used for deflection calculations.

Coefficient used to define the allowable service load tensile stress used for flexural analysis. (Kcc x sqrt(fc), where Kcc = 6 or 12)

The length of bond releasing for the chosen bond released tendons.

Mid-span moment due to self weight of the stub girder.

Mid-span moment due to the dead weight of the members framing onto the stub girder and the weight of the wet cast-in-place slab.

Mid-span moment due to the superimposed dead load.

Mid-span moment due to the superimposed live load.

Moment at point xr due to the dead weight of the members framing onto the stub girder and the weight of the wet cast-in-place slab.

Moment at point xr due to the superimposed dead load.

Moment at point xr due to the superimposed live load.

Moment due at point xe due to the self weight moment of the stub girder (xe = theoretical point of full prestress transfer).

Moment at chosen point of shear investigation for the stubs.

Moment at the chosen point of shear investigation for the gaps.

Factored service load moment at point xc (xc = point of calculation of the compressive force in the flange to be used as the horizontal shear design load).
$M_{cr}$ = Estimated moment to cause flexural cracking used in the concrete shear strength calculations for the stubs.

$M_{crG}$ = Estimated moment to cause flexural cracking used in the concrete shear strength calculations for the gaps.

$M_{ui}$ = Factored mid-span moment due to the non-composite stage of loading.

$M_u$ = Factored mid-span moment due to the composite stage of loading.

$N_{floor}$ = Number of floor members framing onto one side of the stub girder

$N_{gap}$ = Number of gaps in the stub girder.

$N_{stub}$ = Number of stubs on the stub girder.

$N_s$ = Total number of prestressing tendons.

$N_{se}$ = Number of effective prestressing tendons at the member ends accounting for bond released tendons, should they be used.

$N_{release}$ = Number of bond released prestressing tendons.

$N_{fricr}$ = Number of stirrups required for horizontal shear transfer at the cast-in-place slab and stub interface according to the shear friction method.

$n$ = Modular ratio of the cast-in-place concrete to the precast concrete, $E_{cip}/E_c$.

$n_{ps}$ = Modular ratio of the prestressing steel to the precast concrete, $E_p/E_c$.

$O_v$ = Strength reduction factor for shear .

$O_{b/Mn}$ = Nominal shear strength of the non-composite stub girder.

$O_{b/Mn}$ = Nominal shear strength of the composite stub girder.

$P_{dsc}$ = Load applied to the stub girder through one double tee stem as a result of the self weight of the double tees and the weight of the wet topping slab.

$P_{dse}$ = Load applied to the stub girder through one double tee stem as a result of the superimposed dead load acting on the double tees.

$P_{sl}$ = Load applied to the stub girder through one double tee stem as a result of the superimposed live load acting on the double tees.

$P_{st}$ = Factored load applied to the stub girder through one double tee stem as a result of the self weight of the double tees and the construction live load.

$P_i$ = Initial prestress force applied to the stub girder immediately after transfer.

$P_e$ = Effective prestress force applied to the stub girder after all prestress losses.

$P_{ie}$ = Initial prestress force applied to the ends of the stub girder, accounting the reduced number of effective strands, should bond releasing be used.

$P_{ee}$ = Effective prestress force applied to the ends of the stub girder, accounting the reduced number of effective strands, should bond releasing be used.

$Q_o$ = Self weight of the double tee per unit floor area occupied by the double tee.

$Q_{cep}$ = Uniform floor load resulting from the cast-in-place topping slab.

$Q_{dsc}$ = Superimposed dead load per unit floor area.

$Q_{sl}$ = Superimposed live load per unit floor area.

$Q_{con}$ = Construction live load per unit floor area.

$R$ = Estimated prestress losses taken as a percentage of the initial prestressing.

$r_{pi}$ = Reinforcement ratio of the non-composite cross-section at the gap regions.

$r_p$ = Reinforcement ratio of the composite cross-section at the gap regions.

$S_{1c}$ = Non-composite section modulus for the tops of the stubs.

$S_{2c}$ = Bottom non-composite section modulus.

$S_{1ce}$ = Composite section modulus for the tops of the stubs.

$S_{2ce}$ = Bottom composite section modulus.

$S_{3ce}$ = Composite section modulus for the top of the cast-in-place slab.

$S_{max}$ = Maximum allowable stirrup spacing for the stub regions as defined by the ACI Code.

$S_{max}$ = Maximum allowable stirrup spacing for the gap regions as defined by the ACI Code.
\[ S_{\text{web}} = \] Required stirrup spacing for vertical shear.
\[ S_{\text{comp}} = \] Required stirrup spacing for horizontal shear transfer between the stubs and the cast-in-place slab for composite action.
\[ S_{r} = \] Stirrups spacing required for vertical shear in the stub regions.
\[ S_{rg} = \] Stirrup spacing required for vertical shear in the gap regions.
\[ S_{\text{trial}} = \] Trial spacing used for stirrups in the stub region.
\[ S_{\text{trialg}} = \] Trial spacing used for stirrups in the gap region.
\[ S_{\text{fricr}} = \] Required stirrup spacing by shear friction method for the transfer of horizontal forces between the cast-in-place slab and the stubs for composite action.
\[ S_{\text{reqs}} = \] Minimum required spacing in the stub regions based on the requirements of both horizontal shear, vertical shear, and the maximum allowable stirrup spacing (Smax).
\[ S_{\text{reqg}} = \] Required stirrup spacing in the gap regions based on the requirements of vertical shear and the maximum allowable stirrup spacing (Smaxg).

**Stubbars**
- Number of supplementary top reinforcement bars required in the first stub.

**Stub**
- Length of the stubs of the stub girder.

**Tu**
- Maximum factored torsional moment acting on the stub girder.

**Tnc**
- Nominal torsion resistance of the concrete at the section of interest along the stub girder.

**Tfs**
- Top tension force to be carried by the supplementary top reinforcement in the first stub.

**Tfg**
- Top tension force to be carried by the supplementary top reinforcement in the first gap.

**Tfc**
- Top tension force to be carried by the supplementary top reinforcement at mid-span.

**tc**
- Thickness of the cast-in-place compression flange between stubs.

**ta**
- Thickness of the cast-in-place compression flange above the top of the stubs.

**ts**
- Thickness of the cast-in-place slab over the double tees.

**uc**
- Coefficient of friction for a concrete to concrete interface.

**Vu**
- Horizontal shear force used for shear reinforcement design at the cast-in-place slab and stub interface (\( V_u = M_c / (d_{st} - a_{cip} / 2) \)).

**Vut**
- Factored shear force which acts simultaneously with Tu at point xt.

**Vtor**
- Factored shear force calculated at point xt which causes a torsional moment about the centerline of the girder.

**Vx**
- Shear force calculated at point x resulting from the factored superimposed dead and live load.

**Vox**
- Shear force calculated at point x resulting from the self weight of the stub girder.

**Vux**
- Shear force calculated at point x resulting from the factored service loads.

**Vxg**
- Shear force calculated at point xg resulting from the factored superimposed dead and live load.

**Voxg**
- Shear force calculated at point xg resulting from the self weight of the stub girder.

**Vuxg**
- Shear force calculated at point xg resulting from the factored service loads.

**V1ci**
- Estimated strength of the concrete at the stubs against flexure-shear cracking.

**V2ci**
- Minimum flexure-shear cracking strength of the concrete at the stubs.

**V3ci**
- Maximum allowable flexure-shear strength of the concrete at the stubs.

**Vci**
- Flexure-shear strength of the concrete at the stubs used for design (\( V_{ci} = \) smaller of V1ci and V3ci, but not less than V2ci).

**Vcw**
- Web-shear cracking strength of the concrete at the stubs.
**Vc** = Nominal shear strength of the concrete at the stubs  
(Vc = smaller of Vci and Vcw).

**Vs** = Shear strength provided by the stirrup reinforcement at the stubs.

**Va** = Nominal shear strength of the stubs (Vn = Ov (Vs + Vc)).

**V1cig** = Estimated strength of the concrete at the gaps against flexure-shear cracking.

**V2cig** = Minimum flexure-shear cracking strength of the concrete at the gaps.

**V3cig** = Maximum allowable flexure-shear strength of the concrete at the gaps.

**Vcig** = Flexure-shear strength of the concrete at the gaps used for design  
(Vcig = smaller of V1cig and V3cig, but not less than V2cig).

**Vcwg** = Web-shear cracking strength of the concrete at the gaps.

**Vcg** = Nominal shear strength of the concrete at the gaps  
(Vcg = smaller of Vcig and Vcwg).

**Vsg** = Shear strength provided by the stirrup reinforcement at the gaps.

**Vng** = Nominal shear strength of the gaps (Vng = Ov (Vsg + Vcg)).

**Wo** = Uniform load acting on the stub girder as a result of the self-weight of the girder.

**Wdc** = Uniform load acting on the stub girder as a result of the dead weight of the members framing onto the girder and the weight of the wet cast-in-place slab.

**Wdcc** = Uniform load acting on the stub girder as a result of the superimposed dead load.

**Wl** = Uniform load acting on the stub girder as a result of the superimposed live load.

**Wlg** = Portion of the uniform live load acting on the stub girder which is applied to the gross cross-section for deflection calculations.

**Wlcr** = Portion of the uniform live load action on the stub girder which is applied to the cracked cross-section for deflection calculations.

**Wtor** = Portion of the uniform load acting on the stub girder which causes a torsional moment about the girder centerline.

**Wt** = Average self-weight of the stub girder per unit length.

**Wtot** = Total weight of the stub girder.

**Wcip** = Length of the double tee top flange that must be removed and the double tee stems that must be coped at each end of the double tees for the placement of an adequately deep cast-in-place compression flange.

**wp** = Reinforcing index for the non-composite stub girder at the gap regions  
(wp = rpi * fpsl / fc).

**wp** = Reinforcing index for the composite stub girder at the stub regions  
(wp = rp * fps / fc).

**XBay** = Center-to-center spacing of the columns in the direction of the stub girders.

**x** = Member section of interest for the shear reinforcement design of the stubs.

**xg** = Member section of interest for the shear reinforcement design of the gaps.

**xe** = Theoretical point of full prestress transfer (by ACI Code xe = 50 * Ds).

**xs** = Short side of the concrete section found at point xt.

**YBay** = Center-to-center spacing of the columns in the direction of the floor members.

**Yt** = Variable which holds the value of a portion of the calculation carried out to evaluate the nominal torsional strength of chosen concrete section  
(Yt = sqrt(1 + 10 * Pe / Acg * 1 / fc),

**Ypi** = Mid-span deflection of the stub girder due to the initial prestress force.

**Ype** = Mid-span deflection of the stub girder due to the effective prestress force.
\(Y_o\) = Mid-span deflection of the stub girder due to the self-weight of the stub girder.

\(Y_{dc}\) = Mid-span deflection of the stub girder due to the dead weight of the members framing onto the stub girder and the weight of the wet cast-in-place slab.

\(Y_{dce}\) = Mid-span deflection of the stub girder due to the superimposed dead load.

\(Y_l\) = Mid-span deflection of the stub girder due to the superimposed live load.

\(Y_{al}\) = Allowable deflection affection the non-structural elements.

\(Y_{short}\) = Short-term mid-span deflection of the stub girder.

\(Y_{long}\) = Long-term mid-span deflection of the stub girder.

\(Y_{crit}\) = Mid-span deflection affecting the non-structural elements.
Willem J. van Zyverden was born in Pittsburgh, Pennsylvania, on February 16, 1969, the son of Gerald Dirk van Zyverden and Nina Hyatt van Zyverden. After graduating from the Academy of the New Church High School, Bryn Athyn, Pennsylvania, in 1988, he entered Penn State University, State College, Pennsylvania. He received the degree of Bachelor of Science in Civil Engineering from Penn State University in May, 1992. In September, 1992, he entered the Graduate School of Lehigh University.
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