ACI 318-08
Building Code
Requirements for Structural Concrete
October 27, 2009
ICE BOATING
History

**1904** – "Joint Committee on Reinforced Concrete" established with representatives from:

- American Society for Testing and Materials - ASTM
- American Society of Civil Engineers - ASCE
- American Railway Engineering and Maintenance of Way Association (later to become AREA)
- Association of American Portland Cement Manufacturers (later to become PCA).

Initial funding came from the USGS.

In the first years of existence the “Joint Committee” produced several drafts for a Concrete Code.

**1905** – National Association of Cement Users founded (later to become ACI)
1909 – ACI established a committee to develop a Concrete Code later to become ACI Committee 318.

1914 – The "Joint Committee on Reinforced Concrete" published a Concrete Design Code.

1920’s – Both the Joint Committee Requirements and ACI 318 Requirements coexist and become more similar as time passes.

1930’s – At the end of this decade ACI 318 Requirements become the single document in the US dealing with concrete design.
Development of the ACI 318 flexural requirements

1914 - Both the Joint Committee Requirements and the National Association of Cement Users (later to become ACI) used WSD (Working Stress Design) in their codes.

1930’s – The Joint Committee and ACI requirements were merged in a single document named ACI 318 using WSD.

1956 - An Appendix was introduced in ACI 318 for USD (Ultimate Strength Design)

1963 - ACI 318-63 contained both WSD and USD requirements and the designer could choose either method.

1971 - ACI 318-71 was full USD and the WSD was sent to an Appendix.
Development of the ACI 318 flexural requirements

1989 – The name of ACI 318 is changed from “Building Code Requirements for Reinforced Concrete” to “Building Code Requirements for Structural Concrete”.

1995 – The Unified Design Procedure is introduced as an Appendix.

2002 – WSD is taken out completely from ACI 318
ICE FISHING
318 -08

- Meant to be used as a legally adopted building code
- Written in mandatory language
- Minimum requirements for public health and safety
- Deemed to satisfy ISO 19338:2007(E)
- Does not replace sound engineering knowledge, experience, and judgment
318 Does Not

- Govern concrete piles, drilled piers unless in Seismic Design Categories D, E, and F
- Govern slabs on ground
- Govern tanks and reservoirs
- Govern houses – ACI 332
Becoming Law

- Must be adopted in a State Building Code
- Federal government does not adopt/enforce building codes
Chapter 1306 Adopts International Building Code

MINNESOTA BUILDING CODE

Chapter 1300  Administration of the State Building Code
Chapter 1301  Building Official Certification
Chapter 1302  Construction Approvals
Chapter 1303  Minnesota Provisions of the State Building Code
Chapter 1305  Adoption of the 2006 International Building Code
Chapter 1306  Special Fire Protection Systems (optional)
Chapter 1307  Elevators and Related Devices
Chapter 1309  Adoption of the 2006 International Residential Code
IBC 2006
Chapter 19 Adopts ACI 318

Section 1901
General

1901.1 Scope. The provisions of this chapter shall govern the materials, quality control, design and construction of concrete used in structures.

1901.2 Plain and reinforced concrete. Structural concrete shall be designed and constructed in accordance with the requirements of this chapter and ACI 318 as amended in Section 1908 of this code. Except for the provisions of Sections 1904 and 1920, the design and construction of slabs on grade shall not be governed by this chapter unless they transmit vertical loads or lateral forces from other parts of the structure to the soil.
Chapter 19 Adopts ACI 318
ACI 318-08

- Chapter 1 – General Requirements
- Chapter 2 – Notation and Definitions
- Chapter 3 – Materials
- Chapter 4 – Durability Requirements
- Chapter 5 – Concrete Quality, Mixing, and Placing
- Chapter 6 – Formwork, Embedments, and Construction Joints
ACI 318-08

- Chapter 7 – Details of Reinforcement
- Chapter 8 – Analysis and Design-General Considerations
- Chapter 9 – Strength and Serviceability Requirements
- Chapter 10 – Flexure and Axial Loads
- Chapter 11 – Shear and Torsion
- Chapter 12 – Development and Splices of Reinforcement
ACI 318-08

- Chapter 13 – Two-Way Slab Systems
- Chapter 14 – Walls
- Chapter 15 – Footings
- Chapter 16 – Precast Concrete
- Chapter 17 – Composite Concrete Flexural Members
- Chapter 18 – Prestressed Concrete
Chapter 19 – Shells and Folded Plate Members

Chapter 20 – Strength Evaluation of Existing Structures

Chapter 21 – Earthquake-Resistant Structures

Chapter 22 – Structural Plain Concrete
ACI 318-08

- Appendix A – Strut-And-Tie Models
- Appendix B – Alternative Provisions for Reinforced and Prestressed Concrete Flexural and Compression Members
- Appendix C – Alternative Load and Strength Reduction Factors
- Appendix D – Anchoring to Concrete
- Appendix E – Steel Reinforcement Information
1st Case Study

- Klein & Hoffman
- Sixteen Story C-I-P Hotel
- Low Seismic Area / No Special Detailing
- Flat Plate with Shear Walls and Perimeter Beams
- Mat Foundation
2nd Case Study

- Cagley & Associates
- Six Story C-I-P Parking Garage
- Low Seismic Area / No Special Detailing
- Post-Tensioned Beam and Slab for Gravity and Lateral
- Caissons
3rd Case Study

- S.K. Ghosh & Associates
- Six Story C-I-P Parking Garage
- High Seismic Area
- Gravity Post-Tensioned Beam and Slab
- Special Moment Frame
- Piles
### Project Information – Building Description

<table>
<thead>
<tr>
<th>Description</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td>This building was designed by Klein and Hoffman, Chicago, Illinois, and David Fennell supplied the information presented. Some of the original design information was slightly adjusted for ease of presentation. The building was constructed in the Chicago area.</td>
<td></td>
</tr>
</tbody>
</table>

### Structural Framing

- **Systems:**
  - **Gravity:**
    - Flat slab and perimeter beam
    - Vertical supports: Columns and shear walls
    - Lateral:
    - North-South direction: Shear walls
    - East-West direction: Shear walls at elevator core combined with beam-column frames at the outer edges of the floor plate.
  - **Foundations:** Mat foundation
- **Other:** The exterior columns, exterior walls, and spandrel beams are part of the facade, exposed to the elements.

The architecture of this building required wide exterior columns and deep perimeter beams which significantly stiffened the structure in the East-West direction. The end walls and the elevator core were used as shear walls in the North-South direction.

### Building Geometry

- **Height:**
  - Overall: 16 stories, 181 ft 3 in.
  - First story: 26 ft 0 in.
  - Typical floor-to-floor: 9 ft 0 in.
  - Top story: 16 ft 5 in.

- **Plan:**
  - North-South direction: Overall, 66 ft 9 in.
  - Column spacing alternates between 2 spans @ 32 ft 2 in. and spans of 21 ft 2 in. and 43 ft 2 in.
  - East-West direction: Overall, 253 ft 8 in.
  - Interior column spacing varies from 25 ft 9 in. to 30 ft 0 in.
  - Exterior column spacing, typically 13 ft 7 in.

At interior columns, the architect omitted alternating columns to create open floor space. The resulting column layout resembled a diamond pattern in plan, as shown on Sheet S1.02.
###Project Information – Building Codes and Standards

####GENERAL BUILDING CODES

<table>
<thead>
<tr>
<th>Code / Standard</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Governing Building Code:</strong></td>
<td>The local building code that has jurisdiction over the project was consulted before the analysis and design stages to ascertain the overall requirements for the project. The local code referenced in this presentation is the 2006 IBC.</td>
</tr>
<tr>
<td>- Adopted: 2006 International Building Code (IBC)</td>
<td></td>
</tr>
<tr>
<td>- No exceptions by the local jurisdiction</td>
<td></td>
</tr>
<tr>
<td><strong>Supporting Standards:</strong></td>
<td>In this presentation, the local building code is assumed to adopt the 2006 International Building Code (IBC). In turn, the 2006 IBC adopts by reference the 2005 edition of <em>Minimum Design Loads for Buildings and Other Structures</em> (ASCE 7-05) [Section 1601.1], and the 2005 edition of <em>Building Code Requirements for Structural Concrete</em> (ACI 318-05) [Section 1901.2].</td>
</tr>
<tr>
<td>- Building Code Requirements for Structural Concrete (ACI 318-05)</td>
<td>Requirements set forth in these codes were used to analyze, design, and detail the reinforced concrete structural members in this building. Unless noted otherwise, referenced chapter, section, and equation numbers are from ACI 318-05.</td>
</tr>
<tr>
<td>- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)</td>
<td>The provisions of Section 11.1 define the authority and limits of ACI 318-05. Provisions 11.1.1, 11.1.2, 11.1.3, and 11.1.8.3 define the relationship between ACI 318-05 and other codes or standards. The remaining provisions of this section define the limits.</td>
</tr>
<tr>
<td></td>
<td>The codes and standards utilized in a project were listed in the General Notes on the structural drawings (see Sheet 50.00).</td>
</tr>
</tbody>
</table>

####OCCUPANCY / BUILDING USE

| Occupancy classification, 2006 IBC: R-1                                      | R-1 is defined as residential occupancy where the occupants are primarily transient in nature (less than 30 days), including hotels (Section 310.1, 2006 IBC). Occupancy is typically determined by the owner or architect. |
| Occupancy category, ASCE 7-05: II                                            | The occupancy classification was used for determining the occupancy category defined in Table 1-1, ASCE 7-05. The occupancy category was needed to determine the importance factor used in calculating loads such as flood, wind, |
### FIRE RATING

The project required a fire rating of 2 hours. The provisions of Section 721.2.3 of the 2006 IBC govern. Specified cover depends on the fire-resistance rating, the type of aggregate used in the concrete mixture, and whether the member is restrained or unrestrained. The cover requirements in 318-05 for fire resistance are in 7.2.7. In general, the minimum cover requirements of 7.2 - Concrete protection for reinforcement, result in at least a 2-hour fire-resistance rating for the structural members, which is adequate for this occupancy. Member sizes and cover chosen provide at least a 2-hour fire resistance, which meet the requirements of this job.

### PROJECT SPECIFICATIONS

The ACI specifications incorporated into the project specifications were:
- Specifications for Structural Concrete ([ACI 301-05](http://www.concrete.org/TKC/Hotel/SSL/FlashHelp/1_Project_Information/1B_Building_Codes_and_Standards.htm[9/20/2009 11:29:05 AM]))

Project specifications give types of materials to be used and the construction practices to be followed. Klein and Hoffman project specifications typically incorporate industry specifications and other references that are applicable to the project.

The ACI guides and standards modified for use in these project specifications were:
- Mass Concrete ([ACI 307.1R-05](http://www.concrete.org/TKC/Hotel/SSL/FlashHelp/1_Project_Information/1B_Building_Codes_and_Standards.htm[9/20/2009 11:29:05 AM]))
- Guide for Measuring, Mixing, Transporting, and Placing Concrete ([ACI 304R-00](http://www.concrete.org/TKC/Hotel/SSL/FlashHelp/1_Project_Information/1B_Building_Codes_and_Standards.htm[9/20/2009 11:29:05 AM]))
- Guide for Concrete Inspection ([ACI 311.4R-05](http://www.concrete.org/TKC/Hotel/SSL/FlashHelp/1_Project_Information/1B_Building_Codes_and_Standards.htm[9/20/2009 11:29:05 AM]))
- Details and Detailing of Concrete Reinforcement ([ACI 315-99](http://www.concrete.org/TKC/Hotel/SSL/FlashHelp/1_Project_Information/1B_Building_Codes_and_Standards.htm[9/20/2009 11:29:05 AM]))
- Guide to Formwork for Concrete ([ACI 347-04](http://www.concrete.org/TKC/Hotel/SSL/FlashHelp/1_Project_Information/1B_Building_Codes_and_Standards.htm[9/20/2009 11:29:05 AM]))

Klein and Hoffman usually supplements specifications with the information from ACI guides and standards, as needed, to meet individual project requirements. Non-standardized ACI documents, such as reports and guides, are written in permissive language, intended for guidance in planning, designing, executing, and inspecting construction, and commonly discuss several available approaches.

Individuals need to evaluate the significance and limitations of the report’s content and recommendations. When Klein and Hoffman chooses to use information in these documents, the information is written in mandatory language in the project specifications. Although not used as part of these project specifications, [ACI 336.2R](http://www.concrete.org/TKC/Hotel/SSL/FlashHelp/1_Project_Information/1B_Building_Codes_and_Standards.htm[9/20/2009 11:29:05 AM]) also provides valuable information concerning mat foundations.

### STANDARDS REFERENCED IN ACI 318

Section 3.8 lists standards referenced in ACI 318-05. Per provision 3.1.2, materials shall meet the ASTM standards listed in 2.8.1. Several ASTMs are referenced in the drawing notes for this project (see Field Requirements and Material Properties).

There were several other 318-referenced standards utilized for this project, such as 3.8.2, 3.8.6, and 3.8.7; however, they are not discussed within this case study.

### INFORMATION REQUIRED PER ACI 318

ACI 318 requires certain information be provided on plans and specifications. Provision 1.3.1 lists the information required to be on the plans and specifications.
### Records Required per ACI 318

<table>
<thead>
<tr>
<th>Requirement</th>
<th>ACI 318-05</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318-05 requires that certain information to be retained for a given</td>
<td>requires maintaining certain records: project drawings and specifications</td>
</tr>
<tr>
<td>time period after the project is complete. The local jurisdiction had no</td>
<td>(12.1), structural calculations (12.2), and material test records (12.1).</td>
</tr>
<tr>
<td>additional requirements.</td>
<td></td>
</tr>
</tbody>
</table>

### Legal Authority

<table>
<thead>
<tr>
<th>Authority</th>
<th>Building Official's authority—11.8.3; 12.1; and 12.1.</th>
</tr>
</thead>
<tbody>
<tr>
<td>The local jurisdiction gives authority to Building Officials. ACI 318-05</td>
<td></td>
</tr>
<tr>
<td>has provisions that state the authority of Building Officials relative to</td>
<td></td>
</tr>
<tr>
<td>the construction of structural concrete.</td>
<td></td>
</tr>
</tbody>
</table>
16 Story Hotel / Flat Slab / Shear Wall / Mat Foundation

Project Information – Loads and Effects – Gravity Loads

**Loads/Effects**

DEAD LOAD (SERVICE LOAD):
- Self-weight was calculated by the analysis program assuming concrete with a unit weight of 150 pcf (see Methods and Assumptions).
- Superimposed dead load (all floors): 8 psf
- Glass curtain wall load: 12 psf

References and Commentary

Service dead loads include the weight of the structural members, the curtain wall, and other identified loads permanently attached to the building, such as mechanical equipment. Weights of the structural members were computed using assumed member sizes.

A superimposed uniform dead load of 8 psf was applied to typical floors. This load accounts for the weight of miscellaneous mechanical, electrical, and plumbing equipment, as well as other miscellaneous items that are attached to the structure.

**LIVE LOADS (SERVICE LOAD):**

<table>
<thead>
<tr>
<th>AREA</th>
<th>LIVE LOAD (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical floors (Partitions)</td>
<td>40 (Add 20 psf)</td>
</tr>
<tr>
<td>Corridors</td>
<td>80</td>
</tr>
<tr>
<td>Public space</td>
<td>100</td>
</tr>
<tr>
<td>Terraces</td>
<td>100</td>
</tr>
<tr>
<td>Stairs</td>
<td>100</td>
</tr>
<tr>
<td>Mechanical floors</td>
<td>150</td>
</tr>
</tbody>
</table>

Live loads are a function of occupancy or use. The minimum live loads are given in Table 1607.1 of the 2006 IBC. These loads are noted in the General Notes on Sheet 50.00.

A 20 psf partition load (classified as a live load according to IBC) was applied on typical floors, in addition to the 40 psf live load, based on occupancy requirements for this building. This load is slightly greater than the minimum 15 psf partition load specified in Section 1607.5 of the 2006 IBC.

We used live load reductions per Section 1607.9.1 of the 2006 IBC; see Load Combinations and Patterns.
16 Story Hotel / Flat Slab / Shear Wall / Mat Foundation

Project Information – Loads and Effects – Deflection Limits

<table>
<thead>
<tr>
<th>Loads/Effects</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LATERAL DEFLECTION LIMITS</strong></td>
<td></td>
</tr>
<tr>
<td>Wind:</td>
<td>Maximum overall deflection at the top of the building due to service wind forces was much less than the limiting value. While not prescribed by codes, a deflection limit equal to the height of the building divided by 500 is commonly used for buildings of this type.</td>
</tr>
<tr>
<td>• Overall Building Deflection:</td>
<td></td>
</tr>
<tr>
<td>o Limits H / 500 = 181.25 ft x 12 in./ft / 500 = 4.4 in.</td>
<td></td>
</tr>
<tr>
<td>o Maximum calculated deflection at top of building due to wind in N-S direction = 0.2 in. &lt; 4.4 in.</td>
<td></td>
</tr>
<tr>
<td>Seismic:</td>
<td>Interstory drift due to the seismic forces in each story was determined to be less than ASCE 7-05 code-prescribed maximum allowable story drift in each direction.</td>
</tr>
<tr>
<td>• Interstory drift:</td>
<td></td>
</tr>
<tr>
<td>o Limit (per Table 12.12-1 of ASCE 7-05) = 0.02 x story height = 0.02 x 9.67 ft x 12 in./ft = 2.3 in.</td>
<td></td>
</tr>
<tr>
<td>o Maximum calculated drift due to seismic forces in N-S direction = 0.1 in. &lt; 2.3 in.</td>
<td></td>
</tr>
<tr>
<td><strong>GRAVITY DEFLECTION LIMITS</strong></td>
<td></td>
</tr>
<tr>
<td>Roof and floor member deflection limits for service gravity loads:</td>
<td>Deflection limits for various members are given in 1604.3 of IBC 2006, and Table 9.5(b) of ACI 318-05.</td>
</tr>
<tr>
<td>• Immediate deflection due to live load = $\ell / 360$</td>
<td></td>
</tr>
<tr>
<td>• Long-term deflections due to sustained loads plus immediate deflection due to any additional live load = $\ell / 480$.</td>
<td></td>
</tr>
<tr>
<td>Deflection requirements in 9.5 were satisfied for the flat slab and spandrel beams (9.5.1). Deflections were computed by the analysis software in addition to moments, shears, and axial loads.</td>
<td></td>
</tr>
<tr>
<td>Deflection checks:</td>
<td></td>
</tr>
<tr>
<td>• Flat slab</td>
<td></td>
</tr>
<tr>
<td>• Spandrel beam</td>
<td></td>
</tr>
</tbody>
</table>
## Structural Analysis – Member Sizing

### FLAT SLAB

<table>
<thead>
<tr>
<th>Member properties:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness ( (h) ): 8 in. uniform, no drop panels</td>
</tr>
<tr>
<td>( f'_c ): 5000 psi</td>
</tr>
</tbody>
</table>

In general, a preliminary slab thickness is determined based on deflection limits 9.5.3 (9.5.3.1) and two-way shear stress limits when no column-line beams are present (11.12.2).

After providing several options to the contractor, it was determined that an 8-in. thick flat plate slab (no drop panels) would be optimum. The slab deflection was checked against the deflection limits of Table 9.5(b) per 9.5.3.4 (9.5.3.4 allows a reduction of the minimum thickness requirements of Table 9.5(c), see also 9.5.3.2).

Based on the 8 in. slab, a normalweight concrete mixture with a compressive strength of 5,000 psi was specified. The adequacy of this compressive strength is verified during the design stage.

### BEAMS

<table>
<thead>
<tr>
<th>Member properties:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spandrel beams:</td>
</tr>
<tr>
<td>Width ( (b) \times ) overall height ( (h) ): 12 in. ( \times ) 24 in.</td>
</tr>
<tr>
<td>Beams at elevators:</td>
</tr>
<tr>
<td>Width ( (b) \times ) overall height ( (h) ): 12 in. ( \times ) 18 in.</td>
</tr>
<tr>
<td>( f'_c ): 5000 psi</td>
</tr>
</tbody>
</table>

Usually, the depth of a beam is based on deflection requirements (9.5.2, one-way construction [nonprestressed]).

The size of the spandrel beams in this building was dictated by architectural considerations, as the beams are incorporated into the facade. The 24-in. depth specified by the architect was more than sufficient to satisfy deflection requirements for the 15 ft maximum span length (Table 9.5(b)), considering the glass curtain wall is supported by the spandrel beams.

The beams around the elevator openings were sized based on experience. Beams such as these typically do not carry large loads.

Because a normalweight concrete mixture with a compressive strength of 5,000 psi was specified for the floor slab, the same strength was specified for the beams.
INTERIOR COLUMNS

Member Properties:
- First Level:
  - Floor-to-floor height: 28 ft 0 in.
  - Plan dimensions: 36 in. x 30 in.
- Other levels:
  - Floor-to-floor height: 9 ft 8 in.
  - Plan dimensions: 36 in. x 18 in.
- $f_c'$:
  - Foundation to 5th floor = 7000 psi
  - 5th floor to roof = 5000 psi

See Column Schedule for more information.

The size of these columns depends on several factors, including the weight of the supported floors, magnitudes of superimposed loads, bending moments, and constructability.

To establish a column size, the factored gravity loads for a typical interior column were run. It is common to size a column using a minimum 1% reinforcement ratio. If, during later design stages, it is determined that strength requirements are not satisfied, it is more cost efficient to increase column strength by adding reinforcement rather than change the column dimensions. Changing column sizes at a later stage in the design can necessitate a reanalysis of the structure and a subsequent redesign of all of the members.

Knowing that the height of the first story is 28 ft 0 in. and that slenderness effects would have to be considered in the design, a 36 in. x 30 in. column with $f_c' = 7,000$ psi was initially chosen for the first story interior columns.

Above the first story, a 36 in. x 18 in. column was used for the entire column stack; the concrete compressive strength and amount of reinforcement were changed over the building height as a function of factored gravity load. Using the same column size wherever possible increases formwork reuse, which is more economical than changing formwork to save a minor amount of concrete. The 36 in. column dimension was therefore kept constant at all floor levels.

EXTERIOR COLUMNS

Member properties:
- First level:
  - Floor-to-floor height: 28 ft - 0 in.
  - Plan dimensions: 60 in. x 16 in.
- Other levels:
  - Floor-to-floor height: 9 ft - 8 in.
  - Plan dimensions: 60 in. x 12 in.
- $f_c'$:
  - Foundation to 5th floor = 7000 psi
  - 5th floor to roof = 5000 psi

The shape and size of the exterior columns in this building were dictated by architectural considerations; these columns, like the spandrel beams, were incorporated into the facade.

Specified concrete compressive strength of the exterior columns was made the same as that for the interior columns for simplicity of construction.

SHEAR WALLS

Member properties:

The shear walls location and length were dictated primarily by the slab...
- Thickness (h):
  - Foundation to 2nd floor = 12 in.
  - 2nd floor to roof = 10 in.
- $f'_{c}$:
  - Foundation to 6th floor = 7000 psi
  - 6th floor to roof = 5000 psi
See Shear Wall Schedule for more information.

**MAT FOUNDATION**

Member properties:
- Plan dimensions: 78.33 ft x 269.5 ft
- Thickness (h): 48 in. uniform, except as required around elevator core
- $f'_{c}$: 4000 psi

After providing several options to the contractor, it was determined that a typical wall thickness of 10 in. would be the most economical, given the layout of the walls in both directions.

Specified concrete compressive strength in the shear walls changed over the height of the building based on strength requirements.

The geotechnical report provided options for a mat foundation or caissons under the columns and walls. After consultation with the contractor and the owner, a mat foundation proved to be more economical.

Plan dimensions of the mat were determined primarily by plan dimensions of the building (see Sheet S1.01).

The thickness of the mat was chosen based on experience and a two-way (punching) shear calculation, which typically controls the thickness of a mat foundation when the soil is not poor. The adequacy of the 48 in. thick mat was checked during the design stage to ensure that flexural and shear strength requirements are satisfied.

A normalweight concrete mixture with a specified compressive strength of 4,000 psi was adequate for the mat foundation.
# Structural Analysis – Evaluation of Results

## General

Short-hand calculations can be completed to check the reasonableness of the analysis results. The following step-by-step hand calculations were performed by Klein and Hoffman to check the software results of their analysis.

Boundary Conditions:
The reactions and deflections from the computer analysis were spot checked and found reasonable.

Stability:
An overall stability check of the building was performed in conformance with Section 1604.4 of the 2006 IBC.

The analysis satisfied the provisions of 8.1.1; that is, the members were proportioned for adequate strength in accordance with applicable provisions of the code using load factors and strength reduction factors specified in Chapter 9.

Gross computer input errors can be found by a few simple checks:
- Using statics, compare applied loads to the building reactions.
- Check the deflections at the top of the building for reasonableness.

It was found that the factor of safety for overturning of the entire structure was adequate, which was expected for a building of this type (relatively heavy and nonslender).

## Check Flat Slab Analysis

**Verify the factored negative moment at the face of column B.2-22 in the north-south direction**

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>The negative factored moment in the 6 ft - 9.5 in. design strip at the face of this column in the N-S direction given by the computer program is 168.9 ft-kips.</td>
<td>The program provided service and factored negative and positive moment values in both directions. The following check provided a high confidence in the accuracy of these values.</td>
</tr>
<tr>
<td>2</td>
<td>To determine if this moment is reasonable, compute the factored moment at this section using the direct design method.</td>
<td>The Direct Design Method (12.6) could not be used to determine the factored bending moments for this slab system, since the limitations of this method are not satisfied (12.6.1). It should, however, provide a rough check on the software results.</td>
</tr>
</tbody>
</table>
| 3   | Factored loads (distributed uniformly):  
  \[ D = 0.100 + 0.008 = 0.108 \text{ ksf} \]  
  \[ L = 0.06 \text{ ksf} \]  
  \[ q_u = 1.2(0.108) + 1.6(0.06) = 0.226 \text{ ksf} \]  | Load combination—Eq. (9-2)  
  Dead load of the 8 in. slab = (8 / 12) x 150 = 100 psf  
  Superimposed dead load = 8 psf  
  Live load = 40 + 20 (partitions) = 60 psf |
Total factored static moment in span:
\[ M_u = \frac{q_e \ell u \ell_n^2}{8} = \frac{0.208 \times 27.16 \times 19.17^2}{8} = 282.0 \text{ ft} \cdot \text{kip} \]

Direct design—Eq. (13-4)
\[ \ell_2 = \text{the typical tributary width} \]
\[ \ell_n = \text{the exterior clear span.} \]
where,
\[ \ell_2 = 27.16 \text{ ft} \]
\[ \ell_n = 8.25 + 14.92 = 110.17 \text{ ft} \]

Thus, total interior negative factored moment = 0.70\( M_u = 197.4 \text{ ft} \cdot \text{kip} \)

Direct design—13.6.3.2
In an exterior span, 70 percent of \( M_u \) is assumed distributed to the interior support of slab systems without beams between interior supports and with edge beams.

\[ M_u = 0.75 \times 197.4 = 148.1 \text{ ft} \cdot \text{kip} \]

Direct design—13.6.4.1
For slabs without interior beams, 75 percent of the total interior negative factored moment is distributed to the column strip.

The factored interior negative moment in the column strip determined by the direct design method (148.1 ft\( \cdot \)kip) is of the same order of magnitude as that determined by the computer program (168.9 ft\( \cdot \)kip).

This provides a reasonable check of the computer program output.

**CHECK SPANDREL BEAM ANALYSIS**

**VERIFY THE FACTORED NEGATIVE AND POSITIVE BENDING MOMENTS AND FACTORED SHEAR FORCE DUE TO GRAVITY LOADS IN A TYPICAL INTERIOR SPANDREL BEAM**

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Negative and positive factored gravity moments in the spandrel beam given by the computer program are 17.9 ft( \cdot )kip and 8.9 ft( \cdot )kip, respectively. Factored gravity shear force at the face of the column is 14.6 kips.</td>
</tr>
<tr>
<td></td>
<td>The load combinations due to combined gravity and seismic forces governed the flexural design of the spandrel beams.</td>
</tr>
<tr>
<td>2</td>
<td>( w_u = (1.2 \times 0.012 \times 9.67) + [(1.2 \times 0.108) + (1.6 \times 0.060)] \times 10.33 + 1.2 \times 0.3 = 2.8 \text{ kips/ft} )</td>
</tr>
<tr>
<td></td>
<td>The tributary width of the beam is assumed to be a constant 10.33 ft, even though it varies along the span length due to the irregular interior column spacing.</td>
</tr>
<tr>
<td></td>
<td>Load combination—Eq. (9-2)</td>
</tr>
<tr>
<td></td>
<td>Factored beam loads:</td>
</tr>
<tr>
<td></td>
<td>• Weight of 8 in. slab = 100 psf</td>
</tr>
<tr>
<td></td>
<td>• Weight of beam = 12 \times 24 \times 0.15/144=300 \text{ lb/ft}</td>
</tr>
<tr>
<td></td>
<td>• Superimposed dead load = 8 psf</td>
</tr>
<tr>
<td></td>
<td>• Weight of glass curtain wall = 12 psf</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>
| 3 | Clear span of beam:  
   \[ \ell_n = 13.58 - (60 / 12) = 8.6 \text{ ft} \] |   |
| 4 | Negative moment at face of interior support =  
   \[ \frac{w_u l_n^2}{11} = 2.8 \times \frac{8.6^2}{11} = 18.8 \text{ ft} \cdot \text{kips} \]  
   (compared to 17.9 ft⋅kips from the computer program) | Factored negative and positive moments were determined using approximate coefficients. |
| 5 | Positive moment in interior span =  
   \[ \frac{w_u l_n^2}{16} = 2.8 \times \frac{8.6^2}{16} = 12.9 \text{ ft} \cdot \text{kips} \]  
   (compared to 8.9 ft⋅kips from the computer program) |   |
| 6 | Shear force at face of column =  
   \[ \frac{w_u l_n}{2} = 12.0 \text{ kips} \]  
   (compared with 14.6 kips from the computer program) | The results from the hand method to compute shear and moment vary from computer program results mainly because a slightly non-conservative uniform tributary area was assumed. However, the comparison indicates that the software results were reasonable and further refinement of the hand calculation was not necessary. |

- Live load = 40 + 20 (partitions) = 60 psf
- Tributary width = \((6.25 + 14.42) / 2 = 10.33 \text{ ft}\)
# Member Design and Detailing – Flat Slab – Flexure Design

## Structural Design

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
</tr>
</thead>
</table>
| 1   | Section properties:  
|     |
|     | • Thickness: h = 8 in.  
|     | • $f'_c = 5000$ psi  
|     | • $f_y = 60$ ksi  
|     | • Strip width at column: b = 81.5 in.  
|     | • d = 6.9 in.  
|     | • Clear cover = 0.75 in.  

Determine the flexural slab reinforcement in the N-S direction at the design strip at column line B.2. See strip width definition in methods and assumptions.

Note that d was different in the two orthogonal directions:  
• N-S direction: $d = h - c_c - d_{cb} 	imes 0.5 = 8 - 0.75 - 0.75 	imes 0.5 = 6.9$ in.  
• E-W direction: $d = h - c_c - d_{cb} 	imes 1.5 = 8 - 0.75 - 0.75 	imes 1.5 = 6.1$ in.

## References and Commentary

### NEGATIVE REINFORCEMENT

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
</tr>
</thead>
</table>
| 2   | The software indicates that the maximum computed negative moment for the design strip was:  
|     | $M_u = 168.9$ ft-kips  
|     | The critical section for the negative reinforcement was at the face of the column (B.2-22).  

The controlling load combination was Eq. 9-2 (9.2.1).  
$1.2D + 1.6L$

The software averaged the moments across the section of the design strip.  

| 3   | Sections were designed assuming the strength reduction factor, $\Phi = 0.9$ for tension-controlled sections, that is, assuming the strain $\varepsilon_t$ in the tensile reinforcement is greater than or equal to 0.005. This assumption was verified later.  

**Strength reduction factor**—9.3.1 and 9.3.2.1  
**Strain**—10.3.4

| 4   | Assuming a tension-controlled section ($\Phi = 0.9$), the required area of steel is $A_s = 5.87$ in.$^2$  

Procedures to design reinforcement for a rectangular cross-section with tension reinforcement only are covered in textbooks and other technical references. These rely on the design assumptions and general principles given in 10.2 and 10.3

**REFERENCE:**  
For more information on two-way slab design, [click here](https://www.pca-web.org) to see a PCA - Time Saving Design Aid.

$A_{s,min} = 0.0018 \times 81.5 \times 8 = 1.17$ in.$^2 \ < 5.87$ in.$^2$ O.K.

Minimum area of two-way slab reinforcement—10.5.4

Bar size selection was based on experience and discussions with contractor.

Try 14-No. 6 bars $A_s = 6.16$ in.$^2 \ > 5.87$ in.$^2$ O.K.
| 5 | Bar spacing = 81.5 / 14 = 5.8 in. < 2 x 8 = 16 in. O.K. | Maximum spacing of two-way slab reinforcement—10.6.2 and 13.3.2 |
| 6 | Verify section is tension-controlled: | The computed reinforcement tension strain (0.0118) at nominal strength exceeds the minimum strain (0.0050) required to consider the section to be tension-controlled. The assumed concrete compression strain is 0.003 at nominal strength (10.3.4). |
|    | \[ \sigma = \frac{A_{sf} f_y}{0.85 f_b} = \frac{6.16 \times 50}{0.85 \times 380} = 1.1 \text{ in.} \] | Section behavior—10.2.2; 10.2.3; and 10.2.7 |
|    | \[ c = \frac{a}{f_f} \beta_f = \frac{1.1}{0.60} = 1.8 \text{ in.} \] | REFERENCE: Mattock, A. H.; Kizing, L. B.; and Hognestad, E., "Rectangular Concrete Stress Distribution in Ultimate Strength Design," ACI JOURNAL, Proceedings V. 57, No. 8, Feb. 1961, pp. 875-928. |
|    | \[ c_t = \left( \frac{0.003}{c} \right) d_t - 0.003 = \left( \frac{0.003}{1.8} \right) \times 6.9 - 0.003 = 0.018 > 0.0050 \] | |
| 7 | Negative Reinforcement: | See detail for top reinforcement placement. |
|    | • Use 14-No. 6 bars centered over column B.2-22. | |

**POSITIVE REINFORCEMENT**

| 8 | The software indicates that the maximum computed positive moment in the span between column lines 22 and 24 is: \( M_u = 36.0 \text{ ft-kips.} \) | The controlling load case is Eq. 9-2 (9.2.1). \[ 1.2D + 1.6L \] |
|    | The required area of steel was calculated \( A_s = 1.18 \text{ in.}^2 \) | The software averaged the moments across the section of the design strip. |
|    | The required area of steel per foot in the 81.5 in. design strip: | \( A_s \) was also checked against \( A_{s, \text{min}} = 1.17 \text{ in.}^2 \) |
|    | \[ 1.18 / (81.5 / 12) = 0.17 \text{ in.}^2 / \text{ft} \] | |
| 9 | Positive Reinforcement: | See detail for bottom reinforcement placement. |
|    | • Use No. 4 @ 12 in. \( (A_s = 0.20 \text{ in.}^2 / \text{ft}) \) | |

**CHECK UNBALANCED MOMENT TRANSFERRED BY FLEXURE**

| 10 | The software indicates that the maximum computed unbalanced moment in the N-S direction at this slab-column connection (B.2-22) is: \( M_u = 74.1 \text{ ft-kips.} \) | In this building, slab edge moments are resisted by the spandrel beams. Unbalanced moment transferred by flexure were checked at interior column locations because there are no column-line beams. |
|    | A fraction of the unbalanced moment \( y_f M_u \) was transferred by flexure within an effective slab width of: \( c_2 + 3h = 18 + (3 \times 8) = 42 \text{ in.} \) centered on the column. | Two-way slab - unbalanced moment—13.5.3.2 |
\[
\gamma_f = \frac{1}{1 + (2/3)\frac{b_1}{b_2}}
\]

where,
\[
\begin{align*}
    b_1 &= c_1 + d = 36 + 6.9 = 42.9 \text{ in.} \\
    b_2 &= c_2 + d = 18 + 6.9 = 24.9 \text{ in.}
\end{align*}
\]

Therefore, \( \gamma_f = 0.53 \)

The unbalanced moment transferred by flexure

\[
= 0.53 \times 74.1 = 39.3 \text{ ft}-\text{kips}
\]

Assuming a tension-controlled section (\( \Phi = 0.9 \)), the area of steel required to resist the 39.3 ft-kip moment is 1.31 in.\(^2\)

\[
A_{s, \text{min}} = 0.0018 \times 42 \times 8 = 0.61 \text{ in.}^2 < 1.31 \text{ in.}^2 \quad \text{O.K.}
\]

Required number of No. 6 bars

\[
= \frac{1.31}{0.44} = 3 \text{ bars}
\]

Based on the spacing of approximately 5.8 in. for the 14-No. 6 bars within the design strip of 81.5 in., more than 3-No. 6 bars were provided within the 42 in. effective slab width.

Thus, requirements for unbalanced moment transferred by flexure were satisfied.

Factors: unbalanced moment—Eq. 13-1 (13.5.3.2)

The dimensions \( b_1 \) and \( b_2 \) were determined from 11.12.1.2.

Procedures to design tension reinforcement for a rectangular cross-section are covered in textbooks and other technical references. These rely on the design assumptions and general principles given in 10.2 and 10.3.

Minimum area of two-way reinforcement—13.3.1 (7.12.2.1)

Area to concentrate reinforcement required to resist unbalanced moment—13.5.3.2; 13.5.3.3; and 13.5.3.4.

If additional reinforcement would have been required, it is typical to call out this reinforcement in a note on the drawing stating the number of bars and that the bars should be evenly distributed across the effective slab width.

**DEFLECTION CHECK**

Immediate deflection due to service live load:

\[
\Delta_l = 0.13 \text{ in.} \leq \ell / 360 = 0.95 \text{ in.}
\]

where, \( \ell = 28 \text{ ft 7 in.} \)

Deflection requirements of 9.5.3 - Two-way construction (non prestressed), were satisfied for the slab (13.1.4). For slabs without interior beams, minimum slab thickness is governed by the provisions of either 9.5.3.2 or 9.5.3.4. Because the 8 in. slab thickness is less than that required by 9.5.3.2, provisions of 9.5.3.4 were satisfied.

The software computed immediate service load deflections. An effective
Long-term deflection due to sustained loads:

\[ \Delta_{LT} = 0.61 \text{ in.} \leq \ell / 480 = 0.71 \text{ in.} \]

Long-term deflections were computed in accordance with 9.5.2.5 and 8.5.2.5. It was found that both the computed immediate and long-term deflections were less than the maximum permissible values given in Table 9.5(b).
Member Design and Detailing – Flat Slab – Shear Design

### Structural Design

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
</tr>
</thead>
</table>
| 1   | Section properties:  
|     | - Thickness: \( h = 8 \) in.  
|     | - \( f'_c = 5000 \) psi  
|     | - \( f_y = 60 \) ksi  
|     | - Strip width at column: \( b = 81.5 \) in.  
|     | - \( d = 6.9 \) in.  
|     | - Clear cover = 0.75 in.  |

| 2   | Service Loads:  
|     | - \( D \), slab = 100 psf  
|     | - \( D \), superimposed = 8 psf  
|     | - \( L \), floor = 40 psf  
|     | - \( L \), partitions = 20 psf  |

Factored Load:  
- \( q_u = 0.226 \) ksf

The controlling load combination was Eq. 9-2 (9.2.1).
\[ 1.2D + 1.6L \]

### CHECK ONE-WAY SHEAR IN THE N-S DIRECTION

3. Determine factored shear force \( V_u \) for one foot wide design strip:  
\[ V_u = 0.226 \text{ ksf} \times 42.67 \text{ ft} / 2 = 4.8 \text{ kips} \]

One-way shear (beam action) is checked at a critical section located at a distance \( d \) from the face of the support (11.12.1.1, 11.1.3, and 11.1.3.1). In this building, one-way shear limits were satisfied at all joints. As a side note, unless slab panels are unusual, one-way shear doesn't govern in two-way slabs, and this check is often omitted by inspection.

For ease of calculation, \( V_u \) at the column was used as a conservative estimate.

Concrete shear strength \( \varphi V_c \) was determined:

4. Strength-reduction factor—9.3.2.3  
Shear strength - General—11.1.1  
Concrete shear strength—Eq. 11-3 (11.3.1.1)

It can be shown that one-way shear requirements were also satisfied for a
<table>
<thead>
<tr>
<th>Step</th>
<th>Formula/Equation</th>
<th>Notes/Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>( qV_e = q2\sqrt{\frac{b_u d}{12\times E/2}} )</td>
<td>( = 0.75 \times 2 \times \sqrt{\frac{5,000}{12\times 6.9}} )</td>
</tr>
<tr>
<td></td>
<td>( = 11.1 \text{ kips} )</td>
<td>0.75 ( \times ) 2 ( \times \sqrt{\frac{5,000}{12 \times 6.9}} )</td>
</tr>
<tr>
<td></td>
<td>( \geq 4.8 \text{ kips} ) O.K.</td>
<td>Critical section parallel to the N-S direction.</td>
</tr>
<tr>
<td>5</td>
<td>From the computer program, the maximum shear and unbalanced moment in the N-S direction were:</td>
<td>The code requires checking two-way shear action in each direction (N-S and E-W) separately. The stress pattern at the column slab joint under the balanced moment from each direction is computed. The direct shear stress is computed ( (V_d/A_c) ) and added to the stress pattern resulting from the fraction of unbalanced moment transferred by eccentricity of shear (11.12.6). This example shows the computation in the N-S direction.</td>
</tr>
<tr>
<td></td>
<td>( V_u = 115 \text{ kips} )</td>
<td>The effects of slab openings closer than ( 10h ) from the columns are given in 11.12.5. A few small openings are located near some of the columns and shear limits were satisfied at these connections.</td>
</tr>
<tr>
<td></td>
<td>( M_u = 74.1 \text{ ft-kips} )</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Determine factored shear force ( V_u ) at the critical section ( (d/2 \text{ from the column face}) ):</td>
<td>The critical section for two-way shear is located at a distance of ( d/2 ) from the face of the column (11.12.1.2). Total factored shear stress at a slab-column connection is computed in accordance with 11.12.6. Two-way shear requirements at edge and corner column joints were not checked in this building because of the deep spandrel beams. However, these requirements were met at interior columns where there are no column-line beams.</td>
</tr>
<tr>
<td></td>
<td>( V_u = 115 - 0.226 \left( \frac{49.9 \times 24.9}{144} \right) = 113 \text{ kips} )</td>
<td>Transfer of moment - eccentric shear—11.12.6.1, 11.12.6.2, and 13.5.3.1</td>
</tr>
<tr>
<td></td>
<td>Note: ( b_1 = 42.9 \text{ in.} ) and ( b_2 = 24.9 \text{ in.} ) (See Flexure Design).</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Determine maximum factored total shear stress ( v_u ) at the critical section:</td>
<td>Equations to determine properties of critical sections at interior, edge, and corner columns are given in textbooks and other technical references (R11.12.6.2).</td>
</tr>
<tr>
<td></td>
<td>( v_u = \frac{V_u + V_d M_u A_c}{A_t} )</td>
<td>Reference: ( A_c )</td>
</tr>
<tr>
<td></td>
<td>Determine properties of the critical section:</td>
<td>More references</td>
</tr>
<tr>
<td></td>
<td>( A_c = 2(b_1 + b_2) d = 2(42.9 + 24.9) \times 6.9 = 936 \text{ in.}^2 )</td>
<td>Factor: unbalanced moment transferred by eccentricity of shear—Eq. 11-39 (11.12.6.1)</td>
</tr>
<tr>
<td></td>
<td>( J_c = \frac{b_1 d b_1 + b_2 d}{3} + d^3 )</td>
<td>( \frac{936 \times 6.9 \times 6.9 + 6.9^3}{3} )</td>
</tr>
<tr>
<td></td>
<td>( \geq 11,700 \text{ in.}^3 )</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>( V_v = 1 - V_r = 1 - 0.53 = 0.47 )</td>
<td></td>
</tr>
</tbody>
</table>
Thus, total factored combined shear stress was:

\[ v_u = \frac{118,000}{686} + \frac{0.47 \times 74,1 \times 12,000}{11,700} \]
\[ = 121 + 36 = 157 \text{ psi} \]

\[ \psi_f = 0.53 \] (See Flexure Design)

| 7 | Nominal shear strength \( v_c \) is the smallest value from:
\[
\left(2 + \frac{4}{5}\right)\sqrt{\frac{E}{f_t}} = \left(2 + \frac{4}{2}\right)\sqrt{\frac{5,000}{269}} = 263 \text{ psi}
\]
\[
\psi_c = \left(\frac{e_d}{d} + 2\right)\sqrt{\frac{E}{f_t}} = \left[\frac{40 \times 6.9}{(42.9 + 34.3)} + 2\right]\sqrt{\frac{5,000}{269}} = 265 \text{ psi}
\]
\[ 4\sqrt{\frac{E}{f_t}} = 4\sqrt{\frac{5,000}{269}} = 283 \text{ psi} \]

Two-way action for slabs—11.12.2 and 11.12.2.1 (Eqs. 11-33 to 11-35)

| 8 | \( \psi v_c = 0.75 \times 283 = 212 \text{ psi} > 157 \text{ psi O.K.} \)

When factored two-way shear stress exceeds the allowable value, several options can be explored. These include increasing slab thickness, column size, or compressive strength of the concrete. In addition, drop panels or shear caps can be added, or shear reinforcement in accordance with 11.12.3 or 11.12.4 can be placed in the slab. In this building, the two-way shear stress limit was satisfied without utilizing any of these measures.

Strength-reduction factor—9.3.1 and 9.3.2.3
Shear strength - general—11.1.1
16 Story Hotel / Flat Slab / Shear Wall / Mat Foundation

Member Design and Detailing – Flat Slab – Detailing

<table>
<thead>
<tr>
<th>Structural Detailing</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FLEXURAL REINFORCEMENT</strong></td>
<td></td>
</tr>
<tr>
<td>The bar lengths shown on the typical floor plan were determined by the computer program based on the moment contours from the finite element analysis of the slab and by the provisions of ACI Chapter 12.</td>
<td>Recall that the slab-column frames are not designed as part of the lateral-force-resisting system; only gravity loads effects are assumed in the analysis. The negative moment reinforcement cut-off points were determined by the computer program in conformance with Chapter 12.</td>
</tr>
<tr>
<td>Minimum embedment and splices lengths for bars are given in the Lap Splice and Embedment Length table on Sheet 50.01.</td>
<td>Assumptions, calculations, and code references are shown on topic, Lap Splices and Embedments, under Chapter, Miscellaneous Design and Detailing.</td>
</tr>
<tr>
<td>Because the reinforcement was detailed per Chapter 12 for the actual moments, the minimum detailing provisions of Chapter 13 do not apply.</td>
<td>Although flexural cut-off points were determined by Chapter 12, the provisions of 13.3 were used in detailing the termination of flexural reinforcement.</td>
</tr>
</tbody>
</table>

---

**SECTION AT DESIGN STRIP AT COLUMN 22-B.2**

NO SCALE

Top Reinforcement at B.2-22:
Use 14-No. 6 bars centered over column B.2-22, as determined in the flexural design.

Size and number of bars were selected based on required reinforcement, spacing limitations, and experience. For top bars, No. 5 bars or larger are usually specified, primarily for resistance to deformation due to construction traffic.
### Top Bar Cut-Off:

From the computer output, the average negative moment across the defined strip decreased to zero at:
- 52 in. north of column centerline
- 37 in. south of column centerline

The minimum extended length of the top bar north of column centerline was the greater of:
- $d = 6.75$ in.
- $12d_b = 12 \times 0.75$ in. = 9 in.
- $\ell_{ab}/16 = 41.17 \times 12 / 16 = 31$ in. (Controls)

Thus, the minimum length of the top bar extending north was:
52 + 31 = 83 in. or 6 ft 11 in.

Use a length of **14 ft 6 in.** and center the bars on the column.

### Spacing limits for reinforcement:

- **Minimum**—7.6.1 and 7.6.4
- **Maximum**—10.6.2 and 13.3.2

### The following provisions for flexural reinforcement placement were met.

**Development of flexural reinforcement:**

- **General**—12.10.2 and 12.10.3
- **Bottom bars**—12.11.1 and 12.11.3
- **Top bars**—12.12.1, 12.12.2, and 12.12.3

Where there are openings in the slab, reinforcement layout conforms to the provisions of 13.4.

**Reference:**

ACI Committee 315, 2004, ACI Detailing Manual, SP-66, American Concrete Institute.
Bottom Reinforcement at B.2-22:
Use No. 4 @ 12 in. for the positive reinforcement, as determined in the flexural design.

Based on several factors, including the contours and values of the positive moments, the minimum area of reinforcement, minimum required length of reinforcement, and simplicity of statement, it was decided to use a continuous mat of uniformly spaced reinforcement at the bottom of the slab in both directions. Additional bottom bars were added where required for strength, and those additional bars are indicated on the Typical Floor Plan on Sheet 51.02.

Development of flexural reinforcement:
- General—12.10.2 and 12.10.3
- Bottom bars—12.11.1 and 12.11.3
PLAN OF BOTTOM BAR LAYOUT IN DESIGN STRIP
AT COLUMN 22-B.2

NO SCALE

SHEAR REINFORCEMENT

Shear reinforcement is not required.  
Minimum shear reinforcement: 11.5.6.1
ICE PALACE
ACI 318
Reorganize and Rewrite

ACI 318-14
Member – based organization
Goals for 318-14

- Easier to Locate Needed Information
- Integrate Seismic with Other Detailing Requirements
- Introduce Building System Behavior
- Introduce Performance Criteria
- Expand Analysis Approaches
- Increase Adaptability for Innovations
Why Reorganize?

- Code Has Tripled in Size Since 1971
- Member Detailing in Several Chapters
- Takes Time to Understand Where Related Information is Located
- Seismic is Stand Alone
- Innovate Concepts Difficult to “Fit In”
Goals for 318-14

- Easier to Locate Needed Information
- Integrate Seismic with Other Detailing Requirements
- Introduce Building System Behavior
- Introduce Performance Criteria
- Expand Analysis Approaches
- Increase Adaptability for Innovations
THE WORD
"SUSTAINABILITY"
DOES NOT APPEAR IN THE 318 CODE
# Project Information – Geotechnical Data

<table>
<thead>
<tr>
<th>Geotechnical Data</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>GENERAL</strong></td>
<td></td>
</tr>
<tr>
<td>A geotechnical investigation of the site determined key soil properties needed</td>
<td>The geotechnical engineer produced a report that summarized the findings of the</td>
</tr>
<tr>
<td>in design of the foundation and for seismic design.</td>
<td>investigation; the report recommended a mat foundation system.</td>
</tr>
<tr>
<td><strong>SITE CLASS</strong></td>
<td></td>
</tr>
<tr>
<td>A site classification, D, was obtained using steps from ASCE 7-05.</td>
<td>The geotechnical investigation found that soil at the site conforms to Site Class D,</td>
</tr>
<tr>
<td></td>
<td>which is a stiff soil with shear wave velocity, standard penetration resistance,</td>
</tr>
<tr>
<td></td>
<td>and undrained shear strength within the limits in ASCE 7-05 Table 20.3-1. This</td>
</tr>
<tr>
<td></td>
<td>classification by the geotechnical engineer was needed to determine the Seismic Design</td>
</tr>
<tr>
<td></td>
<td>Category of the building.</td>
</tr>
<tr>
<td></td>
<td>To acquire the physical properties of the strata, soil borings were taken at a number</td>
</tr>
<tr>
<td></td>
<td>of locations chosen throughout the site. The lengths of the borings extended to a</td>
</tr>
<tr>
<td></td>
<td>depth of 100 ft below the surface, in accordance with ASCE 7-05.</td>
</tr>
<tr>
<td><strong>SOIL PROPERTIES</strong></td>
<td></td>
</tr>
<tr>
<td>Allowable soil bearing pressure = 4000 psf</td>
<td>The allowable soil bearing pressure (service load limit) and modulus of subgrade</td>
</tr>
<tr>
<td>Spring constant, k = 150 pci</td>
<td>reaction, which were provided in the geotechnical report, were needed to design the</td>
</tr>
<tr>
<td></td>
<td>mat foundation.</td>
</tr>
</tbody>
</table>
# Project Information - Loads and Effects - Load Combinations and Patterns

## LOADS

<table>
<thead>
<tr>
<th>Loads / Effects</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Gravitational Loads</strong></td>
<td>Structural members must be designed to resist effects from all applicable loads (8.2.1), which in this case are dead, live, wind, and seismic loads.</td>
</tr>
<tr>
<td>• Service dead loads.</td>
<td></td>
</tr>
<tr>
<td>• Service live loads.</td>
<td></td>
</tr>
<tr>
<td><strong>Lateral Loads</strong></td>
<td>Service loads and seismic forces are determined in accordance with the 2006 IBC, which is assumed to be the general building code that governs design (6.1.2).</td>
</tr>
<tr>
<td>• Service wind forces depicted in Fig. 6.9 of ASCE 7-05 were applied to the building's lateral-force-resisting system in both the N-S and E-W directions.</td>
<td></td>
</tr>
<tr>
<td>• Service wind pressures over the height of the building.</td>
<td></td>
</tr>
<tr>
<td>• Wind forces over the height of the building in each direction.</td>
<td></td>
</tr>
<tr>
<td>• Strength-level seismic forces acting on the building in both the N-S and E-W directions.</td>
<td></td>
</tr>
<tr>
<td>• Distributed seismic forces over the height of the building in both directions.</td>
<td></td>
</tr>
</tbody>
</table>

## LOAD COMBINATIONS

<table>
<thead>
<tr>
<th>Service load combinations:</th>
<th>The service load combinations were different depending on the deflection limit check. See deflection limits for more information.</th>
</tr>
</thead>
<tbody>
<tr>
<td>• These combinations are used when checking deflection limits. The computer program identifies the critical case for this check.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Factored load combinations:</th>
<th>The computer program combined effects from service gravity loads and lateral loads using load combinations in 9.2 - Required strength.</th>
</tr>
</thead>
<tbody>
<tr>
<td>• These combinations are used when computing the required strength of members. The computer program identifies the critical case from all combinations: 9.2.1</td>
<td>Load factors, general—9.1.1, 9.1.2, and 9.1.2</td>
</tr>
<tr>
<td>Effects—9.2.2 and 9.2.3</td>
<td></td>
</tr>
</tbody>
</table>

## LOAD PATTERNS

<table>
<thead>
<tr>
<th>Slab and Spandrel Beam Analysis</th>
<th>Live load patterns were applied for both service and factored load combinations.</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Two live load patterns - Full live load applied on two adjacent spans and on alternate spans.</td>
<td>Arrangement of load—8.9.1 and 8.9.2</td>
</tr>
</tbody>
</table>

## LIVE LOAD REDUCTION

<table>
<thead>
<tr>
<th>Live load reduction in accordance with Section 1607.9.1 of the 2006 IBC was applied in the summation of gravity loads for the columns and walls.</th>
<th>Live load reduction—6.2.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live load was not reduced for slab design.</td>
<td></td>
</tr>
</tbody>
</table>
## Project Information – Loads and Effects – Lateral Loads

<table>
<thead>
<tr>
<th>Loads / Effects</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>WIND LOADS (SERVICE LEVEL)</strong></td>
<td></td>
</tr>
<tr>
<td>Design factors:</td>
<td>Section 1609.1.1 of the 2006 IBC requires that wind forces be determined by provisions in Chapter 6 of ASCE 7-05. The analysis program generated wind loads at each floor level in a manner consistent with the ASCE 7-05 method.</td>
</tr>
<tr>
<td>• Exposure category—B</td>
<td>Based on a visual survey of the site, it was determined that the exposure category is B.</td>
</tr>
<tr>
<td>• 3-Second gust wind speed—90 mph</td>
<td>Windward and leeward wind pressures were determined at each floor level in accordance with the analytical method of ASCE 7-05. Leeward pressure is constant over the entire height of the building. Windward pressure is computed at each floor level, and it is assumed that this pressure is uniformly distributed between mid-story heights above and below the floor level under consideration.</td>
</tr>
<tr>
<td>• Importance factor (I)—1.0</td>
<td></td>
</tr>
</tbody>
</table>

The link below shows the windward and leeward wind pressures acting on the building in either the N-S or E-W directions. These wind pressures were determined in accordance with ASCE 7-05.

**WIND LOAD DISTRIBUTION DIAGRAM**

Wind force at a floor level was computed by multiplying the sum of the windward and leeward pressures at that level by the tributary story height and the length of the building perpendicular to the direction of wind.

For example, for a N-S wind, the total service wind force at the 2nd floor level is:

\[
W_s = (8.2 + 8.8) \times \left(\frac{25 + 9.67}{2}\right) \times \frac{259.67}{1,000} \\
= 81.2 \text{ kps}
\]

<table>
<thead>
<tr>
<th>SEISMIC FORCES (FACTORED LEVEL)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Design parameters:</td>
<td>Seismic forces were computed using the dead (self-weight and superimposed) loads, including the 20 psf partition load in accordance with the 2006 IBC.</td>
</tr>
<tr>
<td>• North-South Direction:</td>
<td>Section 1613.1 of the 2006 IBC requires that seismic forces be determined by provisions in ASCE 7-05, excluding Chapter 14 and Appendix 11A.</td>
</tr>
<tr>
<td>o Building frame system - ordinary reinforced concrete shear walls</td>
<td>The geotechnical investigation found that soil at the site conforms to Site</td>
</tr>
<tr>
<td>o R = 5</td>
<td></td>
</tr>
</tbody>
</table>
### 3. Lateral Loads

- **Moment frames and ordinary reinforced concrete shear walls**
  - **R = 4.5**
  - **Site class (for soil): D**
  - **Design spectral acceleration, short period (S_DS): 0.19g**
  - **Design spectral acceleration, one second period (S_D1): 0.10g**
  - **Importance factor (I): 1.0**
  - **Seismic use group: I**
  - **Seismic design category: B**
    - No special (seismic) reinforcement detailing is required for any of the structural members in this building.
  - **Method of analysis:** Equivalent Lateral Force Procedure
  - **Seismic base shear:** V (N-S direction) = 817 kips

The link below shows the factored seismic forces acting on the building in the N-S direction. These forces were determined in accordance with ASCE 7-05.

[SEISMIC LOAD DISTRIBUTION DIAGRAM](#)

---

**Class D.**

The seismic base shear, a function of the total dead load of the structure, was distributed over the height of the building at each floor level in accordance with code-prescribed provisions (seismic design category, 1.1.8.3).

Note that the factored seismic forces in the E-W direction are different than the forces, because the seismic-force-resisting system in the E-W direction is different than the one in the N-S direction.
# Project Information – Drawing Notes – Material Properties

## Notes

The General Note sheet contains the following sections:
- **General Notes**
- **Foundation**
- **Mat Notes**
- **Concrete**
- **Abbreviations**

This section highlights the material requirements from these notes.

## References and Commentary

Construction-related information was conveyed to the contractor by means of a note on the structural drawings and through the project specifications.

Specifications were written for this job; however, guidance on specification writing is beyond the scope of this study.

The notes on the General Note sheet are the only specification information presented. The sections below show the applicable provisions used in establishing such notes.

## GENERAL REFERENCES

**ACI SPECIFICATIONS:**

All concrete work shall conform to ACI 301-05, ACI 311.1R-99, ACI 315-95, ACI 318-05, ACI 347-04, and ACI 304R-00.

**ACI SPECIFICATIONS:**

This note requires the contractor to meet ACI requirements for construction. Project specifications typically provide a more detailed listing of required and optional materials. When project specifications are not provided, notes stating that the minimum requirements of selected reference specifications and standards are often used.

## STEEL PROPERTIES

**REINFORCEMENT MATERIAL:**

All reinforcing steel shall be new billet steel, conforming to ASTM A 515 grade 60. All bar detailing and accessories to be furnished shall conform to typical details in the latest ACI Standard 315-99 detailing manual, except as otherwise shown, noted, or specified.

**REINFORCEMENT MATERIAL:**

This steel material meets the following code provisions:
- Requirement for deformed reinforcement—3.5.1
- Specifications for deformed reinforcement—3.5.2 and 3.5.3.1
- Maximum value of reinforcement $f_y$ used in design—2.4 and 11.5.2

## CONCRETE PROPERTIES

**CONCRETE MATERIAL:**

No chlorides shall be allowed in the concrete mixture.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>$f_c$ at 28 days</th>
<th>UNIT WEIGHT</th>
<th>AIR CONTENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mat</td>
<td>4,000 psi</td>
<td>150 pcf</td>
<td>0%</td>
</tr>
<tr>
<td>Frost walls</td>
<td>4,000 psi</td>
<td>150 pcf</td>
<td>4-6%</td>
</tr>
<tr>
<td>Exterior columns and shear walls</td>
<td>See Schedules</td>
<td>150 pcf</td>
<td>4-6%</td>
</tr>
<tr>
<td>Interior columns and shear walls</td>
<td>See Schedules</td>
<td>150 pcf</td>
<td>0%</td>
</tr>
<tr>
<td>Beams and framed slabs</td>
<td>5,000 psi</td>
<td>150 pcf</td>
<td>4-6%</td>
</tr>
<tr>
<td>Interior slab-on-ground</td>
<td>4,000 psi</td>
<td>150 pcf</td>
<td>0%</td>
</tr>
</tbody>
</table>

**CONCRETE MATERIAL:**

This concrete meets the following code provisions:
- Minimum concrete strength—11.1.1
- Cements—3.1
- Aggregates in concrete—3.1
- Water requirements of concrete mixtures—3.4
- Specifications and requirements of admixtures—2.6
- Maximum water/cementitious ratio—4.1.3; Table 4.2.2
- Freezing-and-thawing exposures—4.2; Table 4.2.1
- Aggressive environments—4.4; Table 4.4.1
# Project Information – Drawing Notes – Field Requirements

<table>
<thead>
<tr>
<th>Notes</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td>The general note sheet contains the following sections:</td>
<td></td>
</tr>
<tr>
<td>• General notes</td>
<td></td>
</tr>
<tr>
<td>• Foundation</td>
<td></td>
</tr>
<tr>
<td>• Mat notes</td>
<td></td>
</tr>
<tr>
<td>• Concrete</td>
<td></td>
</tr>
<tr>
<td>• Abbreviations</td>
<td></td>
</tr>
<tr>
<td>This section highlights the field requirements from these notes.</td>
<td>The following field-related notes convey to the contractor the parameters of construction consistent with the building design assumptions.</td>
</tr>
</tbody>
</table>

## CONCRETE WORK

**ACI SPECIFICATIONS:**

All concrete work shall conform to ACI 301-05, ACI 311.1R-99, ACI 315-99, ACI 318-05, ACI 347-04, and ACI 304R-00.

This note requires the contractor to meet the following code provisions:  
- Preparation of equipment—5.7.1  
- Mixing—5.8  
- Conveying—5.9  
- Depositing—5.10  
- Curing—5.11  
- Cold weather—5.12  
- Hot weather—5.13  
- Formwork—6.1, 6.2  

Project specifications typically provide more detailed field and construction requirements and alternatives. When project specifications are not provided, general notes stating that the minimum requirements of selected specifications and standards are often used.

## CONCRETE COVER

**CONCRETE COVER:**

Minimum concrete cover, unless noted otherwise:  
- Uniformed surface in contact with the ground: 3 in.  
- Formed surfaces exposed to earth or weather:  
  - No. 6 bars and larger: 2 in.  
  - No. 5 bars and smaller: 1-1/2 in.  
- Formed surfaces not exposed to earth or weather:  

This cover meets the following 318-05 provisions:  
- Minimum cover requirements—2.2.1  
- Cover requirements for fire resistance—2.7.7
**REINFORCEMENT**

**SHOP DRAWINGS:**
Checked shop drawings showing reinforcing details, including steel sizes, laps, splicing and placement, shall be submitted to the Architect/Engineer for review before fabrication.

This requirement meets the following code provision:
- Drawings and specifications—1.2.1

**CONTINUOUS BARS:**
Continuous reinforcement shall be lapped as follows: top bars near midspans, bottom bars directly over support. Additional laps required for construction shall be class B.

This requirement meets the following code provisions:
- Splices of reinforcement—7.13.2.2, 7.13.2.3, and 12.14.1

**BAR SUPPORTS:**
Provide adequate bolsters, hi-chairs, support bars, etc., to maintain specified clearances for the entire length of all reinforcing bars. Provide continuous No. 4 spacer bars in walls and slabs to support dowels.

Provide plastic-tipped accessories for reinforcement at all faces of exposed concrete, interior or exterior.

This requirement meets the following code provisions:
- Placing Reinforcement—7.5.1, 7.5.2, 7.5.2.1, and 7.5.2.2

**FIELD BENDING OF BARS:**
All field bending of reinforcing shall be done cold. Heating of bars will not be permitted.

The Engineer of Record permitted field bending of bars in this project. This requirement meets the following code provisions:
- Bending—7.3.1 and 7.3.2

**FOUNDATION DOWELS:**
Provide dowels in caisson caps for walls and columns, piers, pilaster, etc. to match reinforcing above.

This requirement meets the following code provisions:
- Transfer of force—15.8.1 and 15.8.2

**CONSTRUCTION JOINTS:**
Submit detailed drawings showing the locations and detailing of all construction joints, curbs, and slab depressions, openings, sleeves, etc. Construction joints shall be provided as shown on drawings and details. No joint shall be omitted or added without the approval of the engineer.

Provide vertical construction joints in exposed concrete walls at a maximum of 35 ft 0 in. intervals. All joints below grade shall be provided with a bentonite waterstop.

This requirement meets the following code provisions:
- Drawings and Specifications—1.2.1
- Depositing—5.10
- Construction joints—6.4
**EMBEDDING AND CONDUITS:**

No aluminum of any type shall be allowed in the concrete work, unless coated to prevent aluminum concrete reaction. Maximum O.D. of embedded conduit shall be no larger than one-third of the slab thickness, located in the middle third of the slab.

No electrical conduit shall be placed above the welded-wire fabric in slabs-on-ground, or above the top reinforcing in a supported slab.

This requirement meets the following code provisions:
- Conduits and pipes embedded in concrete—6.3
# Structural Analysis – Methods and Assumptions

<table>
<thead>
<tr>
<th>Structural Analysis</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td>The structure was analyzed using commercially available finite-element based analysis programs.</td>
<td>To obtain accurate numerical solutions and to reduce engineering computation time, commercial computer programs were utilized to analyze and design the structural members in this building. A computer program that specializes in concrete slab and mat design was used to analyze and design the flat slabs and the mat foundation, and another program was used to analyze and design the beams, columns, and shear walls. Both programs employ a finite element method of analysis, which conforms to the requirements of 8.3 - Methods of analysis (8.3.1; 8.3.2; 8.3.3). Three-dimensional models of the structure were created in both programs. Member stiffness and other criteria were input into the computer models to satisfy appropriate code requirements (both ACI 318 and 2006 IBC) for material nonlinearity and cracking. These modeling criteria are explained in more detail in the following sections.</td>
</tr>
</tbody>
</table>

## General Computer Input

<table>
<thead>
<tr>
<th>LOADS:</th>
<th>See Gravity Loads.</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Service gravity loads were input to the computer program</td>
<td></td>
</tr>
<tr>
<td>- Concrete unit weight = 150 pcf</td>
<td></td>
</tr>
<tr>
<td>- Live load reduction was computed in accordance with Section 1607.9.1 of the 2006 IBC for the columns and shear walls.</td>
<td></td>
</tr>
<tr>
<td>- Wind and seismic loads were generated by the computer program</td>
<td></td>
</tr>
<tr>
<td>- The computer program combined effects from gravity loads and lateral loads using load combinations in 9.2 - Required strength.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MEMBER SIZES:</th>
<th>See Load Combinations and Patterns.</th>
</tr>
</thead>
<tbody>
<tr>
<td>See Member Sizing for information on sizing and concrete compressive strengths.</td>
<td></td>
</tr>
</tbody>
</table>
**CONCRETE PROPERTIES:**
- Modulus of elasticity:
  - $E_c = 57,000 \sqrt{f'_c}$
    - For $f'_c$ of 4000 psi: 3,600,000 psi
    - For $f'_c$ of 5000 psi: 4,030,000 psi
    - For $f'_c$ of 7000 psi: 4,770,000 psi

- Stiffness - Moment of inertia used in the frame analysis:
  - Slab: $0.25I_g$
  - Beam: $0.35I_g$
  - Column: $0.70I_g$
  - Shearwall: $0.35I_g$

**REINFORCEMENT PROPERTIES:**
- Modulus of elasticity, $E_s$: 29,000 ksi
- Specified yield strength, $f_y$ and $f_yt$: 60 ksi

**ASSUMPTIONS: SYSTEMS AND MEMBERS**

**GENERAL:**
The shear walls and the perimeter beam-column frames were designated as the lateral-force-resisting system in the orthogonal directions. The slab was assumed to act as a rigid diaphragm connecting elements of the lateral-force-resisting system. The slab-column frames are assumed to provide no flexural resistance to lateral forces.

**LATERAL-FORCE-RESISTING SYSTEM:**
- Members:
  - North-South direction—Shear walls only
  - East-West direction—Shear walls combined with perimeter beam-column frames.
  - Diaphragm—concrete slab - assumed rigid
- Applied force location:

**Provisions for $E_c$:** 8.5.1

**Provisions for stiffness:** 8.6.1, 9.5.2.3, and 13.5.1.2

The reduced moments of inertia of the members in the structure were taken from 10.11.1.

**Provision for $E_s$:** 8.5.2

**Provisions that limit $f_y$ and $f_yt$:**
- Longitudinal and transverse reinforcement—2.4
- Shear reinforcement—11.5.2
- Torsional reinforcement—11.6.3.4
- Shear friction reinforcement—11.7.6

**Member interaction:** 8.2.3, 8.3.1

Lateral forces are distributed at each level to the lateral-force-resisting system via the rigid diaphragm (the concrete slab). The diaphragm distributes the lateral force to the members of the lateral-force-resisting system in proportion to their relative lateral rigidities.

Wind forces are assumed to act at each floor perpendicular to the building’s face. Because the wind force does not act through the center of rigidity, the
- Wind forces—Applied at each level at the perimeter.
- Seismic forces—Applied at each level at center of mass.

torsion created by the eccentricity between the center of rigidity and the line of action of the wind force was resolved.

Seismic forces, which are directly related to the building's inertia, are assumed to act through the center of mass at each level. Because the building's center of rigidity and its center of mass do not coincide, torsion was created and then resolved.

**FLAT SLAB:**

Due to the irregular column layout, design strips were defined by the engineer based on the plan geometry and the moment contours. The design strips are shown in the links below:

- Design Strip at B.2 in the N/S Direction
- Design Strip at B.2 in the E/W Direction

Factored gravity bending moment contours for bending about the two principle directions obtained from the computer program for the typical floor slab are shown in the links below. The solid black orthogonal lines are user-defined mesh lines that delineate the finite elements in the model. The color scale shows the range of bending moment (in.-kips) within the finite elements.

- Slab moment contours, \( M_{yy} \) (in.-kips)
- Slab moment contours, \( M_{xx} \) (in.-kips)

Because the column layout in this building is not on an orthogonal grid, the direct design method (13.6) was not permitted, and accurately computing the slab factored bending moments and shear forces would be difficult using the Equivalent Frame Method (13.7). It was decided to analyze the slab by the finite-element method using commercially available software. The use of such a procedure is mentioned in 813.5.1. The slab was modeled using shell elements.

To assist the flexural section design and slab reinforcement layout, geometric "strips" are defined. The code-defined column and middle strips were not easily applied to this building's column grid, so design strips were defined based on bending moment contours obtained from the computer output.

Although these strips are not column and middle strips as defined in 13.7, this method of defining strips is permitted, because it satisfies conditions of equilibrium and geometric compatibility (13.5.1).

The geometry for each user-defined design strip in both directions was input into the program, which computed required flexural reinforcement based on the total factored bending moments in each strip at critical sections.

Unbalanced moment transferred by flexure from the slab to the column was checked by the software in accordance with 13.5.3 at all slab-column connections.

The software verified that the factored shear was less than both one-way and two-way shear strengths at all slab-column connections. The two-way shear stress at each slab-column connection was computed by adding the direct shear stress pattern and the stress pattern caused by the fraction of unbalanced moments transferred by eccentricity of shear in the two orthogonal directions. The computed factored shear stress was less than the code limit for the 8-in. slab at each column joint.
**BEAMS:**
The software that analyzed the frame also provided beam moments and shears. It also provided required area of flexural and shear reinforcement at critical (and other) sections.

The required shear and torsion reinforcement was computed by the program in accordance with 11.1, 11.2, 11.5, and 11.6.

The required area of flexural reinforcement was computed in accordance with the design assumptions and general requirements of 10.2 and 10.3. The provided area of flexural reinforcement is not less than the minimum area of steel required by 10.5. The critical section for the negative reinforcement was the face of the supporting column (8.7.2 and 8.7.3).

**COLUMNS:**
The software used to analyze the frames was utilized to determine the factored bending moments, shears, and axial forces in the columns.

Required longitudinal reinforcement was computed in accordance with the design assumptions and general requirements of 10.2, 10.3, and 10.8. Slenderness effects were also considered in the design (10.10). In the computer model, member stiffness properties were taken in accordance with 10.11.1. The area of longitudinal steel provided in the columns conforms to the limits of 10.3. The critical section for the column flexural reinforcement was at the face of the horizontal framing member, that is the beam or slab. The column to foundation connection was modeled as a fixed base.

Columns: 8.6.1, 8.6.2, 8.6.3, and 8.6.4.

**SHEAR WALLS:**
Walls were designed to resist the combined effects of factored axial loads, bending moments, and shear forces.

Required horizontal reinforcement was computed in accordance with the shear provisions for walls in 11.10, and the provisions thereby referenced. The wall to foundation connection was modeled as a fixed base.

Walls - 14.2.1

**MAT FOUNDATION:**
Finite-element method software was used to analyze the mat foundation. The soil was modeled as springs connected to the bottom of the mat using the subgrade modulus from the geotechnical report.

To view the mat moment contours, select link:
- Mat. moment contours: $M_{yy}$ (in-kips)
- Mat. moment contours: $M_{xx}$ (in-kips)

This finite-element model is consistent with that described in 15.10.3. Note that it is not permitted to analyze a mat foundation using the Direct Design Method of 13.6 (15.10.2).

Reactions at the base of the columns and walls, acquired from the analysis of the building frame, were input to the computer program at appropriate locations at the top of the mat.

The average soil pressure due to unfactored loads was computed to be less than the allowable bearing capacity of the soil (R15.10.1).

The computer program combined effects from service loads using load combinations in 9.2 (15.10.1).

To assist the flexural section design and slab reinforcement layout, geometric "strips" are defined, based on plan geometry and bending moment contours.
The software verified that the factored shear was less than both one-way and two-way shear strengths at all mat-column connections. One-way shear (beam action) is computed at a critical section located at a distance \( d \) from the face of the column or wall (11.12.1.1). The two-way shear stress was computed at each column/wall-mat connection based on a three-dimensional analysis of the joint, that is, the total factored shear stress is the direct shear stress added to the stresses resulting from the fraction of unbalanced moments transferred by eccentricity of shear in the two orthogonal directions.

**REFERENCE:**

ACI Committee 336, "Suggested Analysis and Design Procedures for Combined Footings and Mats (ACI 336.2R-98)," American Concrete Institute, Farmington Hills, MI, 1998, 21 sp.

<table>
<thead>
<tr>
<th>STAIRS:</th>
<th>The slab thickness was obtained by dividing the span length in inches by 20, which is applicable to simply supported solid one-way slabs (see Table 9.5(a)). Even though the dowels that are used to tie the stairs to the structure can develop some restraint (as opposed to a true simple span), this effect is ignored when determining thickness.</th>
</tr>
</thead>
<tbody>
<tr>
<td>The concrete stairs on this project were designed as simple span, one-way slabs.</td>
<td>The stair span was taken as the horizontal distance between supports, and live load is applied to the horizontal projection of the stair. The weight of the stair was also applied on the horizontal projection.</td>
</tr>
</tbody>
</table>
**Member Design and Detailing – Spandrel Beam – Torsion Check**

<table>
<thead>
<tr>
<th>Structural Design</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>NO.</strong></td>
<td><strong>STEP</strong></td>
</tr>
<tr>
<td>1</td>
<td>The critical section for shear and torsion was at distance, d, from the face of the column.</td>
</tr>
<tr>
<td></td>
<td>Member section properties:</td>
</tr>
<tr>
<td></td>
<td>• Width (b) x height (h): 12in. x 24 in.</td>
</tr>
<tr>
<td></td>
<td>• ( f'_c = 5000 ) psi</td>
</tr>
<tr>
<td></td>
<td>• Clear cover = 1.5 in.</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Maximum factored shear force and torsional moment at the critical section:</td>
</tr>
<tr>
<td></td>
<td>( V_u = 14.6 ) kips</td>
</tr>
<tr>
<td></td>
<td>( T_u = 16.8 ) ft-kips</td>
</tr>
</tbody>
</table>

**THRESHOLD TORSION**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The beam was checked to see if torsion could be neglected. Torsion can be neglected if:</td>
</tr>
<tr>
<td></td>
<td>( T_u &lt; \Psi \sqrt{\frac{f'<em>c}{p</em>{cp}}} \frac{A_{cp}^2}{p_{cp}} )</td>
</tr>
<tr>
<td></td>
<td>The overhanging flange width of slab used in the calculation of ( A_{cp} ) flange width = 16 in. &lt; 4 x 8 = 32 in.</td>
</tr>
<tr>
<td></td>
<td>Section properties considering overhanging flange width:</td>
</tr>
<tr>
<td></td>
<td>( A_{cp} = (24 \times 12) + (8 \times 16) = 416 ) in.²</td>
</tr>
<tr>
<td></td>
<td>( p_{cp} = 2(24 + 12 + 16) = 104 ) in.</td>
</tr>
<tr>
<td></td>
<td>( A_{cp}^2 = 1064 ) in.³</td>
</tr>
<tr>
<td></td>
<td>( p_{cp} = ) 104 in.³</td>
</tr>
<tr>
<td>3</td>
<td>Check section properties to see if flanges should be ignored:</td>
</tr>
<tr>
<td></td>
<td>( A_{cp} = 24 \times 12 = 288 ) in.²</td>
</tr>
<tr>
<td></td>
<td>( p_{cp} = 2(24 + 12) = 72 ) in.</td>
</tr>
<tr>
<td></td>
<td>( A_{cp} ) is the gross area of the section resisting torsion.</td>
</tr>
</tbody>
</table>
\[
\frac{\Lambda_{cp}^2}{P_{cp}} = 1.152 \text{ ft}^3
\]

Use,
\[
\frac{\Lambda_{cp}^2}{P_{cp}} = 1.664 \text{ ft}^3
\]

Thus,
\[
    \phi \sqrt{f_t} \left( \frac{\Lambda_{cp}^2}{P_{cp}} \right) = \frac{0.75 \sqrt{\frac{5,000 \times 1,664}{12,000}}}{29.6} = 7.4 \text{ ft-kips} < 16.0 \text{ ft-kips}
\]

Therefore, torsion was considered.

**MAXIMUM TORSION**

Because this is a statically indeterminate structure where reduction of the torsional moment can occur due to redistribution of internal forces after cracking, the compatibility torsional moment was determined:

\[
T_u = \phi \sqrt{f_t} \left( \frac{\Lambda_{cp}^2}{P_{cp}} \right) = 4 \times 7.4 = 29.6 \text{ ft-kips}
\]

Because the factored torsional moment, 16.8 ft-kips, is between 29.6 ft-kips and the minimum value, torsion reinforcement was determined to resist the factored torsional moment (see Transverse Reinforcement for calculations).

**GEOMETRY CHECK**

The adequacy of cross-sectional dimensions was checked.

For solid sections, the following equation must be satisfied:

Strength reduction factor—9.3.2.2

Torsion reduction - statically indeterminate—11.6.2.2

\[T_u \text{ maximum} - 11.6.2.2(a)\]

Commentary—R11.6.2.1 and R11.6.2.2

Size limit—Eq. 11-18 (11.6.3.1)
\[
\left( \frac{V_u}{b_{wd}} \right)^2 + \left( \frac{T_u p_h}{1.7 A_{oh}} \right)^2 \leq \phi \left( \frac{V_c}{b_{wd}} + 6\sqrt{f'_c} \right)
\]

assume No. 4 stirrups,
\[p_h = 2([12 - 2(1.5 + 0.25)] + [24 - 2(1.5 + 0.25)]) = 58 \text{ in.}\]
\[A_{oh} = [12 - 2(1.5 + 0.25)] \times [24 - 2(1.5 + 0.25)] = 174 \text{ in.}^2\]
Thus,
\[
\left( \frac{V_u}{b_{wd}} \right)^2 + \left( \frac{T_u p_h}{1.7 A_{oh}} \right)^2 = \sqrt{\left( \frac{14.600}{12 \times 21.5} \right)^2 + \left( \frac{16.8 \times 12,000 \times 93}{1.7 \times 174^2} \right)^2} = 234 \text{ psi}
\]
\[\phi \left( \frac{V_c}{b_{wd}} + 6\sqrt{f'_c} \right) = 0.75 \times (2 + 6) \times \sqrt{5,000} = 530 \text{ psi} > 234 \text{ psi}\]
Thus, the cross section is adequate.

REFERENCE:

Note that the term \(V_C/b_{wd}\) is equal to \(2\sqrt{f'_c}\).
Strength reduction factor—9.3.3.3
Concrete shear strength—11.3.1.1
### Member Design and Detailing – Spandrel Beam – Transverse Reinforcement Design

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
<th>Structural Design</th>
<th>References and Commentary</th>
</tr>
</thead>
</table>
| 1   |      | The software indicates that the maximum computed shear and torsional moment were:  
     |      | $V_u = 14.6$ kips  
     |      | $T_u = 16.8$ ft-kips  
     |      | Determine the transverse reinforcement in spandrel beam 3B2. Based on combined factored shear force, factored torsional moment, beam dimensions, material strength, and cover, the required transverse reinforcement was computed in accordance with standard assumptions.  
     |      | Shear strength requirement— 11.1.1  
     |      | Sections used in these steps— 11.3, 11.5; and 11.6. |
| 2   |      | The transverse reinforcement required for torsion was determined:  
     |      | $A_t = \frac{T_u}{s} \cdot \frac{\varphi}{2 A_e f_y t \cot \theta}$  
     |      | $= \frac{16.8 \times 12}{0.75 \times 2 \times 0.065 \times 174 \times 60 \times \cot 45}$  
     |      | $= 0.015$ in.$^2$/in.  
     |      | The ratio $A_t / s$ was derived by combining and rearranging Eq. 11-20 (11.6.3.5) and Eq. 11-21 (11.6.3.6).  
     |      | This equation gives the required area of one leg of a closed stirrup per inch of beam length. |
| 3   |      | The transverse reinforcement required for shear was determined:  
     |      | $A_v = \frac{V_u - \varphi V_c}{q f_y d}$  
     |      | $= \frac{34,600 - \left(0.75 \times 2 \times \sqrt{0.065} \times 12 \times 21.5\right)}{0.75 \times 2 \times 60 \times 21.5}$ $< 0$  
     |      | Therefore, the transverse reinforcement does not need to be designed for shear.  
     |      | The ratio $A_v / s$ was derived by combining and rearranging Eq. 11-1 (11.1.1),  
     |      | Eq. 11-2 (11.1.1), Eq. 11-3 (11.3.1.1), and Eq. 11-15 (11.5.7.2).  
     |      | This equation gives the required area of one leg of a closed stirrup per inch of beam length. |
|     |      | **MINIMUM TRANSVERSE REINFORCEMENT**  
     |     | The minimum transverse reinforcement area was computed:  
     |     | $V_u$ was greater than $\varphi V_c / 2$. Therefore, minimum shear reinforcement was required.  
<pre><code> |     | The minimum transverse reinforcement for the combined shear and torsion was calculated by 11.6.5.1 and 11.6.5.2. |
</code></pre>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
</table>
| 4 | \[
\frac{A_v}{2s} + \frac{A_t}{s} = \frac{0.75 \sqrt[2]{f_v} b_v}{2f_{yv}}
\]
|   | = \[
\frac{0.75 \sqrt[2]{6,000} \times 12}{2 \times 60,000} = 0.005 \text{ in.}^2 / \text{in}
\]; and
|   | \[
\frac{A_v}{2s} + \frac{A_t}{s} = \frac{50 f_v}{2f_{yv}}
\]
|   | = \[
\frac{25 \times 12}{60,000} = 0.005 \text{ in.}^2 / \text{in}
\].
|   | The minimum area of transverse reinforcement was less than that required for torsion (step 2).
| 5 | The maximum spacing of transverse reinforcement was the smallest of:
|   | \[
\left\{ \begin{array}{l}
\frac{P_h}{A} = \frac{50}{8} = 7.3 \text{ in. (Governs)}
\\
1.2 \text{ in.}
\\
\frac{d}{2} = \frac{21.5}{2} = 10.8 \text{ in.}
\end{array} \right.
\]
|   | Maximum spacing of shear and torsion reinforcement—11.5.5.1 and 11.6.6.1
| 6 | Assuming No. 4 closed stirrups, the required spacing at the critical section = 0.2 / 0.015 = 13.3 in. > 7.3 in.
|   | The controlling provision was 11.6.6.1.
| 7 | Provide No. 4 closed stirrups @ 6 in.
|   | A spacing of 6 in. on center for the full length of the beam was chosen for ease of construction and inspection. The closed stirrups were anchored by a 135-degree standard hook per 11.6.4.2.
### Member Design and Detailing – Spandrel Beam – Longitudinal Reinforcement Design

<table>
<thead>
<tr>
<th>Structural Design</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>NO.</strong></td>
<td><strong>STEP</strong></td>
</tr>
<tr>
<td>1</td>
<td>The software indicates that the maximum computed negative moment and torsional moment were:</td>
</tr>
<tr>
<td></td>
<td>( M_u = 17.3 ) ft-kips</td>
</tr>
<tr>
<td></td>
<td>( T_u = 16.8 ) ft-kips</td>
</tr>
</tbody>
</table>

#### DUE TO TORSION

| | |
| 2 | Because the beam was required to resist torsion, the minimum area of longitudinal reinforcement required was: |
| | \[ A_{x,min} = \frac{5\sqrt{3}}{4.15} \frac{A_{cp}}{f_y} = \left( \frac{A_t}{s} \right) \frac{I_{yy}}{t_y} \] |
| | \[ = \frac{5 \times 5,000 \times 4.15}{60,000} = 0.033 \times \frac{58}{58} = 0.54 \text{ in.}^2 \] |
| | One-fourth of this area of longitudinal reinforcement (0.14 in.\(^2\)) needed to be distributed around the perimeter of the closed stirrups. |
| | • Maximum perimeter spacing = 12 in. |
| | • \( d_{b,min} = 0.375 \text{ in.} \) |

#### NEGATIVE REINFORCEMENT DUE TO FLEXURE

| | Beam - critical section— 8.7.1 |
| 3 | Based on the combined factored bending moment, beam dimensions, material strength, and cover at the column face, required flexural reinforcement \( A_b \) was computed in accordance with standard assumptions. |
| 4 | Sections were designed assuming the strength reduction factor \( \Phi = 0.9 \) for tension-controlled sections, that is, assuming the strain \( \varepsilon_t \) in the tensile reinforcement is greater than or equal to 0.005. This assumption was verified later. |

#### Longitudinal reinforcement - Torsion— 11.6.5.1
- Minimum area of reinforcement—Eq. 11-24 (11.6.5.3)
- The transverse reinforcement was determined to be No. 4 @ 6 in. minimum. Therefore, \( A_t / s \) was 0.033.

#### Spacing and size of longitudinal reinforcement for torsion— 11.6.6.2
- \( d_{b,min} = 0.042 \times 6 \text{ in.} = 0.25 \text{ in.} \)
- \( d_{b,min} = 0.375 \text{ in.} \) (governs)

Strength-reduction factor— 9.3.1, 9.3.2.1
Strain— 10.3.4

Procedures to design tension reinforcement for a rectangular cross section are
Assuming a tension-controlled section (Φ = 0.9), the required area of steel is $A_s = 0.18 \text{ in.}^2$

$$A_{s,\text{min}} = \frac{3,500 \cdot 0.012 \cdot 41.5}{60,000} = 0.91 \text{ in.}^2$$

(governs)

$$A_{s,\text{req}} = \frac{200 \cdot 0.012 \cdot 41.5}{60,000} = 0.86 \text{ in.}^2$$

Minimum reinforcement— Eq. 10-3 (10.5.1)

**COMBINE TORSIONAL AND FLEXURAL REINFORCEMENT**

<table>
<thead>
<tr>
<th>Side Bars:</th>
<th>Bar size was based on economy and experience.</th>
</tr>
</thead>
<tbody>
<tr>
<td>One No. 6 bar mid-depth on each side face:</td>
<td></td>
</tr>
<tr>
<td>$A_s = 0.44 \text{ in.}^2 \geq 0.14 \text{ in.}^2$</td>
<td></td>
</tr>
<tr>
<td>Perimeter spacing = 10 in. ≤ 12 in.</td>
<td></td>
</tr>
</tbody>
</table>

**Top and bottom Bars:**

An area of 0.14 in.$^2$ was added to the flexural reinforcement area required at the top and bottom of the section.

<table>
<thead>
<tr>
<th>Combine the minimum area of steel due to negative bending with longitudinal reinforcement required for torsion = 0.91 + 0.14 = 1.05 in.$^2$</th>
<th>The computed reinforcement tension strain (0.0293) at nominal strength exceeds the minimum strain (0.0050) required to consider the section to be tension-controlled (10.3.4).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Try 3-No. 6 bars ($A_s = 1.32 \text{ in.}^2$) &gt; 1.05 in.$^2$</td>
<td>Section behavior— 10.2.2, 10.2.3, and 10.2.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Verify that the section is tension controlled:</th>
<th>Reference: Mattuck, A. H.; Kriz, J. B.; and Hooper, E., 1961. &quot;Rectangular Concrete Stress Distribution&quot;</th>
</tr>
</thead>
</table>
\[
\sigma = \frac{f_y}{f_y} = \frac{1.32 \times 60}{0.69} = 1.8 \text{ in.}
\]
\[
c = \frac{a}{b} = \frac{1.5}{0.6} = 2.5 \text{ in.}
\]
\[
\varepsilon_t = \left( \frac{0.062}{0.002} \right) = 31.5
\]
\[
\left( \frac{0.038}{0.019} \right) > 1.5 - 0.038
\]
\[
0.0299 > 0.0200
\]

Thus, the section is tension controlled.

Requirements for maximum spacing to control flexural cracking are given in 10.6 (10.6.1, 10.6.2, 10.6.6, and 10.6.7).

In a narrow beam (12 in.) with 3 longitudinal bars, the maximum bar spacing was OK by inspection, but the computation is shown for clarity. Note that \(c_t\) is the cover to the longitudinal bars.

Check maximum bar spacing for the top and bottom faces:

\[
s = \frac{40,000}{f_y} \times 0.17 \times 12
\]

where,

\(f_y = (2/3) f_y = 14,000 \text{ psi}\)

and,

\(c = 1.5 + 0.5 = 2.0\) in.

Thus,

\(s = 15 - (2.5 + 2.0) = 10\) in.

Actual bar spacing =

\[
\frac{12 - 2(1.5 + 0.5) - 0.75}{1} = 11.5 \text{ in. OK.}
\]

Check minimum bar spacing:

- Minimum clear spacing = \(d_b = 0.75\) in. < 1.0 in. (governs)
- Actual clear space = \(3.6 - 2.5 \times 0.75 = 1.7\) in. > 1.0 in. OK.

Requirements for minimum spacing are given in 7.6 (7.6.1, and 7.6.2). Note that two and one-half bar diameters were subtracted to account for splicing conditions (7.6.4).

Use 3-No. 6 top bars.

Positive reinforcement for this beam would be determined in a similar fashion, based on the maximum factored positive moment in the span and the required torsional longitudinal steel.

DEFLECTION CHECK

Immediate deflection due to service live load:

\(\Delta_l = 0.008 \text{ in.} \leq \frac{1}{360} = 0.45 \text{ in.}\)

where, \(l = 13\) ft. - 7 in.

Deflection requirements of 9.5.2. One-way construction (non prestressed), were satisfied for the spandrel beams.

Because the spandrel beams directly support a glass curtain wall, immediate and long-term deflections were computed in accordance with 9.5.2.1, 9.5.2.3, 9.5.1.4, 9.5.1.5, and 9.5.2.6. Because the spans were very short, computed deflections were much less than the maximum permissible values given in Table 5.8.10.
### Member Design and Detailing – Spandrel Beam – Detailing

**Structural Detailing**

<table>
<thead>
<tr>
<th>LONGITUDINAL REINFORCEMENT DETAILING IN SPANDREL BEAM 3B2</th>
</tr>
</thead>
<tbody>
<tr>
<td>The longitudinal bar size and spacing were determined in <a href="#">Longitudinal Reinforcement</a>.</td>
</tr>
<tr>
<td>- Top and bottom bars: 3-No. 6 continuous</td>
</tr>
<tr>
<td>- Side bars: 1-No. 3 continuous</td>
</tr>
</tbody>
</table>

After the number and size of longitudinal bars were determined at the critical sections, the lengths and lap locations needed to be determined.

Sections 12.10, 12.11, and 12.12 contain rules for positive and negative reinforcement lengths and cut-off locations. Because these are spandrel beams, the integrity requirements of 7.13.2.2 needed to be met. Because these beams are part of the lateral-load-resisting systems, additional positive moment reinforcement requirements (12.11.2) needed to be met.

Minimum embedment and splices lengths for bars are given in the [Lap Splice and Embedment Length](#) table on Sheet 50.01.

**References and Commentary**

<table>
<thead>
<tr>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td>The size and number of beam longitudinal bars were selected based to meet the required reinforcement, spacing limitations, and other required details of Chapters 7 and 12.</td>
</tr>
</tbody>
</table>

Longitudinal reinforcement is contained in the Beam Schedule on Sheet 52.03.

Assumptions, calculations, and code references are shown on topic, Lap Splices and Embedments, under Chapter, Miscellaneous Design and Detailing.

By providing continuous reinforcement the provisions of structural integrity and flexural development provisions were satisfied.

Development of flexural reinforcement:
- Structural integrity— 7.13.2.2.
- Torsional reinforcement— 11.6.4.3.
- Critical section / termination of bar— 12.10.2, 12.10.3, 12.10.4, and 12.10.5.
- Positive moment reinforcement— 12.11.1, 12.11.2, and 12.11.3.
The stirrup size and spacing were determined in Transverse Reinforcement:

- Stirrup size: No. 4
- Stirrup spacing: 6 in.

By providing a 135° bend, the requirements for closed stirrups were met. The stirrup detailing requirements are illustrated below.

Stirrup detailing provisions:

- Lateral reinforcement—7.11.2
- Structural integrity—7.13.2.3
- Placement details—11.5.4 and 12.13.1
- Details of torsional reinforcement—11.6.4.1 and 11.6.4.2
- Development of web reinforcement—12.13.2 and 12.13.2.1

A summary of the closed stirrups is given in the Beam Schedule on Sheet S2.03.
SPANDREL BEAM B2 - SPLICING AND STIRRUP SPACING
NO SCALE
# Member Design and Detailing – Interior Column – Vertical Reinforcement Design

## Structural Design

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
</tr>
</thead>
</table>
| 1   | Member section properties at first story:  
|     | - Height: 28 ft 0 in.  
|     | - Plan dimensions: 36 in. x 30 in.  
|     | - $f_c'$: 7000 psi  
|     | - Clear cover: 1.5 in. |
|     | Determine the vertical reinforcement for interior column (B.2-22) from the first story to the second story. |
| 2   | The maximum factored load and moment from the analysis in the N-S direction were:  
|     | $P_u = 2,415$ kips; and  
|     | $M_u = 48.2$ ft-kips (bottom of column) |
|     | The controlling load combination was Eq. 9-2 (9.7.1).  
|     | $1.2D + 1.6L$ |

## Estimate the Vertical Reinforcement

| 3   | Minimum amount of reinforcement is $0.01A_g$:  
|     | $A_{st} = 0.01 \times 36 \times 30 = 10.8$ in.$^2$  
|     | Try 12-No. 9 bars ($A_{st} = 12.0$ in.$^2$); evenly spaced around the perimeter. |
| 4   | Determine column axial strength with ties:  
|     | $\varphi P_{n,max} = 0.80\varphi[0.65f'_c(A_g - A_{st}) + f_yA_{st}]$  
|     | $= 0.80 \times 0.65[0.65 \times 7(1,080 -12) + (60 \times 12)]$  
|     | $= 3,679$ kips $> P_u = 2,415$ kips |
|     | Check OK. |
| 5   | Minimum area of steel and number of bars in compression members— 10.9.1 and 10.9.2 |

## Calculate the Moment Magnification

| 6   | The column was determined to be nonsway per Eq. 10-6 (10.114.2) |
| 7   | The controlling load combination for determining stability was Eq. 9-4 (9.7.1),  
|     | $1.2D + 0.5L + 1.6W$ |
| 8   | The stability index, $Q$, for the first story was determined: |
\[ Q = \sum P_u \Delta_u \frac{V_{us}}{V_{us}} \times \varepsilon \]
\[ = \frac{70,200 \times 0.05}{1,370 \times (28 \times 12)} \times 0.01 \times 0.00 \]
\[ = 0.01 \times 0.00 \]

Thus, the first story frame was nonsway.

\( \Sigma P_u \) and \( \Sigma V_{us} \) are the total factored vertical load and the horizontal story shear, respectively, in the story being evaluated. \( \Delta_u \) is the first-order relative lateral deflection between the top and bottom of the story due to \( V_{us} \). For calculation of the these variables, click here.

REFERENCE:

MORE REFERENCES:
Slenderness effects were considered per Eq. 10-7 (10.12.2):
Provisions:
- Effective length facto, \( k \rightarrow 10.12.1 \)
- Radius of gyration, \( r \rightarrow 10.11.2 \)
- Unsupported length, \( \ell_n \rightarrow 10.11.3.1 \)

Note:
\( M_1 = (1.2 \times 12.3) + (1.6 \times 1.7) = 17.5 \text{ ft-kips} \)
\( M_2 = (1.2 \times 29.6) + (1.6 \times 7.9) = 48.2 \text{ ft-kips} \)

and, the term \([34-12 \frac{M_1}{M_2}]\) shall not be taken greater than 40.

The slenderness ratios were determined. Slenderness effects may be ignored if:
\[
\frac{k \ell_n}{r} < 34 - 12 \frac{M_1}{M_2} \\
\frac{1.0 \times [(28 \times 12) - 8]}{0.3 \times 36} \leq 34 - 12 \left(\frac{17.5}{48.2}\right) \\
30.4 \leq 29.5
\]

Thus, slenderness effects needed to be considered.

The factored amplified moment \( M_c \) used as input to the column interaction chart was determined:
\[ M_c = \delta_{ns} M_2 \]

where,
\[ \delta_{ns} = \frac{C_m}{1 - 0.75P_c} \]
\[ = \frac{0.75}{1 - 2.415} = 1.12 \]
\[ \frac{0.75 \times 9.779}{0.75 \times 2.415} \]

\[ M_{2min} = P_u (0.6 + 0.03h) = 2.415 [0.6 + (0.03 \times 36)] / 12 = 338.1 \text{ ft-kips (controls)} > M_2 = 48.2 \text{ ft-kips} \]

Thus,
\[ M_c = 1.12 \times 338.1 = 378.7 \text{ ft-kips} \]

The magnified moment was determined by Eq. 10-8 (10.12.3).

The moment magnifier was determined by Eq. 10-9 (10.12.3).
- \( C_m = 0.75 \) per Eq. 10-13 (10.12.3.1). For calculation, click here.
- \( P_c = 9,779 \text{ kips} \) Eq. 10-10 (10.12.3). For calculation, click here.

Reference:

\( M_2 \) was checked against the moment due to "minimum" eccentricity per Eq. 10-14 (10.12.3.2).

INTERACTION DIAGRAM

The interaction diagram for this column is shown below and the point

This diagram was created using basic assumptions for strength design.
\( P_{u} = 2415 \text{ kips}, \quad M_{c} = 379 \text{ ft-kips} \) is plotted.

It is clear that the 12-No. 9 bars are adequate for combined axial and flexural strength requirements. A similar analysis was performed in the E-W direction, where the slenderness ratio was larger. The column is adequate for design in the E-W direction as well.

Methods used to draw interaction diagrams are covered in textbooks and other technical references (10.2 and 10.3).

REFERENCE:
For more information on column design, click here to see a PCA - Time Saving Design Aid.
### Member Design and Detailing – Interior Column – Transverse Reinforcement Design

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
<th>References and Commentary</th>
</tr>
</thead>
</table>
| 1   | From the gravity analysis, the maximum factored shear force in the column in either direction in the first story was: $V_u = 3$ kips | Check the shear strength of interior column (B.2-22) from the first story to the second story. The controlling load combination was Eq. 9-2 (9.2.1).
1.2D + 1.6L. |
| 2   | Determine shear strength based on the shear strength of the concrete only: $qV_e = 2a\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_b b_u d}$ | Shear forces were considered in the design of transverse reinforcement in columns (11.1.1). Because the interior columns were not designated as part of the lateral-force-resisting system, the shear force transferred to the columns was assumed to be due to gravity loads only. |
|     | $= 2 \times 0.75\left(1 + \frac{2,415,000}{2000 \times 36 \times 30}\right)\sqrt{7,000} \times 36 \times 2561000$ | Strength reduction factor—9.3.2.3
Shear strength—11.1.1 and Eq. 11-4 (11.3.1.2). |
|     | $= 247$ kips $> V_u = 3$ kips O.K. | It is clear that a 3 kip shear force is trivial. The presentation is made to show how to apply the code in this case. Shear strength was computed using $d$ in the direction of the least column dimension. |

where $N_u$ is the factored axial compressive force on the column (lbs)
### 16 Story Hotel / Flat Slab / Shear Wall / Mat Foundation

**Member Design and Detailing – Interior Column – Detailing**

<table>
<thead>
<tr>
<th>Structural Detailing</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>VERTICAL (LONGITUDINAL) REINFORCEMENT DETAILING</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Vertical Reinforcement:</strong></td>
<td>The area of longitudinal steel provided in the columns conforms to the minimum and maximum requirements of <strong>10.9.</strong></td>
</tr>
<tr>
<td>The longitudinal bar size and spacing for B.2-24 was determined in Vertical Reinforcement:</td>
<td>Size and number of bars were selected based on required reinforcement, spacing limitations, and experience.</td>
</tr>
<tr>
<td>• 12-<strong>No. 9 bars</strong>, evenly distributed around the perimeter (first story).</td>
<td>Minimum number of bars— <strong>10.9.2.</strong></td>
</tr>
<tr>
<td>Longitudinal reinforcement is summarized in the Column Schedule.</td>
<td>Minimum bar spacing— <strong>7.6.3</strong> and <strong>7.6.4</strong>.</td>
</tr>
<tr>
<td></td>
<td>Assumptions, calculations, and code references are shown in Lap Splices and Embedments.</td>
</tr>
<tr>
<td>Minimum embedment and splices lengths for bars are given in the Lap Splice and Embedment Lengths table on Sheet 50.01.</td>
<td>Vertical reinforcement was spliced using lap splices conforming to <strong>12.17.2.2</strong>. Such splices were located immediately above each floor level.</td>
</tr>
<tr>
<td><strong>Column Splice Detail:</strong></td>
<td>• Tension lap splices— <strong>12.15</strong> (see Lap Splices and Embedments for example)</td>
</tr>
<tr>
<td>All column splices have Class B tension lap splices, which meet the requirements of 12.15 through 12.17. Note that a tension lap splices shall not be less than 30d_b (compression splice).</td>
<td>• Compression lap splices— <strong>12.16</strong> (see Column to Footing Connection for example)</td>
</tr>
<tr>
<td>The typical Interior Column Splice Detail is shown on sheet 52.01.</td>
<td>• Special lap splices in columns— <strong>12.17.1</strong></td>
</tr>
<tr>
<td></td>
<td>• Lap splices in columns— <strong>12.17.2</strong></td>
</tr>
<tr>
<td>Horizontal support of offset bends was provided by column ties at bend. Ties shall resist 1.5 times the horizontal component of the computed force in the inclined portion of the offset bar. The horizontal component was determined for the capacity of the bar at maximum incline, 1 in 6 (9.5° from vertical).</td>
<td>Offset bent longitudinal bars conforming to <strong>7.8.1</strong> were provided at each lap splice location, unless noted otherwise.</td>
</tr>
<tr>
<td>The bar capacity was determined:</td>
<td>• Maximum slope— <strong>7.8.1.1</strong></td>
</tr>
<tr>
<td>$P_u = \varphi P_{n} = 0.85 \varphi f_y A_{st} = 0.85 \times 0.8 \times 1.0 \text{ in.}^2 \times 60 \text{ ksi} = 40.8 \text{ kips}$</td>
<td>• Geometry of longitudinal bar— <strong>7.8.1.2</strong></td>
</tr>
<tr>
<td>1.5 times the horizontal component was determined:</td>
<td>• Horizontal support at bend— <strong>7.8.1.3</strong></td>
</tr>
<tr>
<td>$P_{u, \text{horz}} = 1.5 \times 40.8 \times (1 / 6) = 10.2 \text{ kips}$</td>
<td>• Maximum offset— <strong>7.8.1.5</strong></td>
</tr>
<tr>
<td>The number of ties were determined, assuming #3 ties:</td>
<td>Strength reduction factor— <strong>9.3.2.2(b)</strong></td>
</tr>
<tr>
<td>Compressive limits—Eq. 10-2 (<strong>10.3.6.2</strong>).</td>
<td></td>
</tr>
</tbody>
</table>
No. of ties = \( \frac{P_{u,hor}}{\omega f_y A_y} \) = \( \frac{10.2}{0.9 \times 0.11 \times 60} \) = 1.7

Therefore, 2 ties minimum per offset within 6 in. of bend.

**SECTION AT COLUMN B.2/22 - SPLICE AT 2ND FLOOR**

**COLUMN TIE DETAILING**

<table>
<thead>
<tr>
<th>Confinement Requirements:</th>
<th>Transverse tie reinforcement details conform to the provisions of 7.10.5.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse reinforcement was shown on Sheet S2.01 for the different tie arrangements.</td>
<td>Minimum tie size— 7.10.5.1</td>
</tr>
<tr>
<td>Minimum tie bars size is No. 3 for longitudinal bars No. 10 or smaller.</td>
<td>A maximum 12 in. tie spacing was used for all of the columns (both interior and exterior), and was based on requirements for the exterior columns.</td>
</tr>
<tr>
<td>Use No. 3 ties</td>
<td>Maximum vertical tie spacing— 7.10.5.2</td>
</tr>
<tr>
<td>Maximum vertical tie spacing is the lesser of:</td>
<td>Minimum horizontal tie spacing— 7.10.5.3</td>
</tr>
<tr>
<td>( 16d_{h, long} = 16 \times 1.00 = 16 \text{ in.} )</td>
<td>First tie location— 7.10.5.4</td>
</tr>
<tr>
<td>( 48d_{h, tie} = 48 \times 0.375 = 18 \text{ in.} )</td>
<td></td>
</tr>
<tr>
<td>( M_{h}\left(c_1, c_2\right) = 18 \text{ in.} )</td>
<td></td>
</tr>
<tr>
<td>Use 12 in. maximum spacing.</td>
<td></td>
</tr>
<tr>
<td>A tie was provided at every longitudinal bar for lateral support. See tie configuration for interior columns in figure below.</td>
<td></td>
</tr>
<tr>
<td>Maximum to distance to first tie above foundation or slab shall be less than one half the tie spacing.</td>
<td></td>
</tr>
</tbody>
</table>
- Provide No. 4 ties @ 3 in. at column ends; 12 in. balance

Shear Requirements:
Because the factored shear was less than 50% of the nominal concrete shear strength of the column, shear reinforcement was not required.

Other requirements for tie for confinement of reinforcement:
- Confinement at joint—7.9.1 and 7.9.2
- Confinement in Compression Members—7.10.1

Minimum shear reinforcement—11.5.6.1

30"x36"

18"x36"
### Member Design and Detailing – Shear Wall – Horizontal Reinforcement Design

#### Structural Design

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Member section properties at first story:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Height: 28 ft 0 in.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Plan dimensions: 245 in. x 12 in.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- $f'_c$: 7000 psi</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Clear cover: 1.0 in.</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>The maximum factored shear from the analysis in the N-S direction was:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_u = 579$ kips</td>
<td></td>
</tr>
</tbody>
</table>

#### WALL SHEAR STRENGTH

- The wall's nominal concrete shear strength was determined:
  \[
  \psi V_c = \psi f' c (\frac{f_c}{1000} x 12 \times 196) 
  \]
  \[
  = 0.75 \times 2 \times \sqrt{\frac{7,000}{1000}} \times 12 \times 196 
  \]
  \[
  = 295 \text{ kips} < V_u = 579 \text{ kips} 
  \]
Thus, horizontal shear reinforcement was required.

- Strength reduction factor—9.3.2.3
- Strength limit—11.10.5
- Definition, \( d \)—11.10.4
  - \( d = 0.8 \ell_w = 0.8 \times 245 = 196 \text{ in} \)

#### SHEAR REINFORCEMENT

- Rearranging equations to compute area of steel per unit height of wall:
  \[
  \frac{A_v}{s} = \frac{V_u - \psi V_c}{\psi f'_c d} 
  \]
  \[
  = \frac{579 - 295}{0.75 \times 60 \times 196} = 0.032 \text{ in.}^2 / \text{in.} 
  \]
- The ratio $A_v/s$ was derived by combining and rearranging Eq. 11-1 (11.1.1), Eq. 11-2 (11.1.1), and Eq. 11-31 (11.10.9.1).

- Determine the spacing for required area of steel, assuming 2 - No. 4 bars (one each face of wall):
  \[
  s = \frac{2 \times 0.20}{0.032} = 12.5 \text{ in.} 
  \]
- Try No. 4 @ 12 in. each face.

- Two layers of reinforcement are used in all walls (14.3.4).
Maximum spacing was the smallest of:

\[
\ell_w / 3 = 245 / 3 = 82 \text{ in.}
\]
\[
3 \ell_b = 3 \times 12 = 36 \text{ in.}
\]
\[
\text{18 in. (governs)}
\]

Maximum bar spacing—11.10.9.3 and 14.3.5.

Determine minimum reinforcement:

\[
\rho_s = \frac{2 \times 0.2}{12 \times 12} = 0.0026 > 0.0025 \text{ O.K.}
\]

The area of horizontal steel provided in the walls conforms to the minimum requirements of 14.3.1, 14.3.2, and 11.10.9.2.

Compute wall nominal shear strength:

\[
V_n = V_c + V_s \leq 10\sqrt{f'c}\ell_b h d
\]

\[
= 394 + \frac{2 \times 0.2 \times 60 \times 196}{12} \leq 10 \sqrt{7,000} \times 12 \times 196
\]

\[
= 786 \text{ kips} \leq 1,968 \text{ kips O.K.}
\]

\[
\varphi V_n = 0.75 \times 786 = 590 \text{ kips} \geq V_u = 579 \text{ kips}
\]

Horizontal Reinforcement:

- Use No. 4 @ 12 in. each face

Maximum nominal shear—11.10.3.
### Member Design and Detailing – Shear Wall – Vertical Reinforcement Design

#### Structural Design

<table>
<thead>
<tr>
<th>NO.</th>
<th>STEP</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Member section properties at first story:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Height: 28 ft 0 in.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Plan dimensions: 245 in. x 12 in.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• $f'_{c}$: 7000 psi</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• $\rho_t = 0.0028$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Clear cover: 1.0 in.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Determine the vertical reinforcement for the shear wall, SW-7, on Line aa, from the first story to the second story.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Required vertical reinforcement for each wall was computed by the computer program in accordance with the design assumptions and general requirements of 10.2, 10.3, 10.10, 10.11, 10.12, 10.13, 10.14, 10.17, 14.2, and 14.3 (14.4).</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>The factored loads and moments for the controlling load combinations from the analysis in the N-S direction:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P_u = 3,028$ kips, $M_u = 0$, and $V_u = 0$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P_u = 2,564$ kips, $M_u = 9,906$ ft-kips, and $V_u = 579$ kips</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P_u = 1,394$ kips, $M_u = 9,906$ ft-kips, and $V_u = 579$ kips</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Load combination, Eq. 9-2 (9.2.1): $1.2D + 1.6L$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Load combination, Eq. 9-4 (9.2.1): $1.2D + 0.5L + 1.6W$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Load combination, Eq. 9-6 (9.3.1): $0.9D - 1.6W$</td>
<td></td>
</tr>
</tbody>
</table>

#### REQUIRED MINIMUM VERTICAL REINFORCEMENT FOR SHEAR

| 3   | The minimum required vertical reinforcement was determined:  |
|     | $\rho_v = 0.0025 + 0.5 \left( 2.5 - \frac{h_w}{\varepsilon_w} \right) \left( \rho_t - 0.0025 \right)$  |
|     | $= 0.0025 + 0.5 \left( 2.5 - \frac{181.25}{20.41} \right) \left( 0.0028 - 0.0025 \right)$  |
|     | $= 0.0015 < 0.0025$  |
|     | Thus, minimum for $\rho_v = 0.0025$ for shear  |
|     | Minimum vertical reinforcement for shear— Eq. 11-32 (11.10.9.4)  |
|     | Note for Eq. 11-32, $h_w$ is the overall height of the shear wall, 181.25 ft, and $\varepsilon_w$ is the overall width of the shear wall, 245 / 12 = 20.42 ft.  |

#### VERTICAL REINFORCEMENT REQUIRED FOR FLEXURE AND AXIAL LOADS

| 4   | The capacity of the shear wall was determined in the N-S direction. The shear wall in the N-S direction was nonsway with no slenderness effects.  |
|     | The calculations for the sway and slenderness check in the N-S direction and the design in the E-W direction have been omitted for the sake of brevity. These calculations are very similar to the Exterior Column calculations.  |
| 5   | Maximum spacing of vertical reinforcement was the smallest of:  |
|     | Maximum bar spacing— 11.10.9.5 and 14.3.5  |
Try No. 5 @ 12 in. each face vertical:

\[ \rho_v = \frac{2 \times 0.31}{12 \times 12} = 0.0043 > 0.0025 \quad \text{O.K.} \]

Layers of reinforcement in walls—14.3.4
Minimum vertical reinforcement—14.3.1 and 14.3.2

The interaction diagram contains the points corresponding to the load combinations. No. 5 @ 12 in. were clearly adequate to resist the combined axial load and bending moments.

In accordance with 14.4 (14.2.2), walls subject to axial load or combined flexure and axial load are designed as compression members, like columns. Thus, an interaction diagram of the wall section is needed to determine strength requirements.

Methods used to draw interaction diagrams are covered in textbooks and other technical references (10.2 and 10.3).

This diagram was created using basic assumptions for strength design, assuming the entire wall cross section as a compression member.

Because the aspect ratio (height to length) of the wall is low, it was expected that low reinforcement ratio would be sufficient.

Reinforcement details for the shear walls are given on Sheet S2.02.

Increased vertical reinforcement size and number was provided at the wall ends, to achieve a greater nominal moment strength.

Fourteen No. 6 bars were used at the wall ends, 4 ft 6 in. (length "L" on the wall schedule).
**16 Story Hotel / Flat Slab / Shear Wall / Mat Foundation**

### Member Design and Detailing – Shear Wall – Detailing

<table>
<thead>
<tr>
<th>Structural Detailing</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reinforcement Wall, SW-7:</strong></td>
<td>Two layers of reinforcement are used in all walls (14.3.4).</td>
</tr>
<tr>
<td>• Horizontal Reinforcement: #4 @ 12 in., each face</td>
<td></td>
</tr>
<tr>
<td>• Vertical Reinforcement: #5 @ 12 in., each face</td>
<td></td>
</tr>
<tr>
<td>o Use 14-No. 6, each end</td>
<td></td>
</tr>
<tr>
<td><strong>Ties:</strong></td>
<td>Because the total vertical reinforcement in the wall is less than 0.01 multiplied by the gross concrete area of the wall, vertical reinforcement need not be enclosed by lateral ties (14.3.6).</td>
</tr>
<tr>
<td>Check ( p &lt; 0.01 ) on gross area of wall:</td>
<td></td>
</tr>
<tr>
<td>( A_p = (2 \times 14 \times 0.44) + (0.31 \times 22) = 19.1 \text{ in.}^2. )</td>
<td></td>
</tr>
<tr>
<td>( p &lt; 19.1 / (12 \times 245) = 0.0065 &lt; 0.01 \text{ OK.} )</td>
<td></td>
</tr>
<tr>
<td>• No. 3 ties @ 12 in. at end bars (No. 6 only)</td>
<td>Lateral ties are provided around each of the larger vertical bars at the ends of the wall to help confine the concrete. The size and spacing of these ties match those of the horizontal reinforcement in the walls.</td>
</tr>
<tr>
<td><strong>Splicing in Wall:</strong></td>
<td>Assumptions, calculations, and code references are shown in Lap Splices and Embedments.</td>
</tr>
<tr>
<td>Vertical and horizontal reinforcement are spliced using lap splices conforming to 12.15. Vertical reinforcement splices are located immediately above each floor level.</td>
<td></td>
</tr>
<tr>
<td>Minimum embedment and splices lengths for bars are given in the Lap Splice and Embedment Lengths table on Sheet S0.01.</td>
<td></td>
</tr>
</tbody>
</table>
Openings in Wall:
Provide a minimum of 2-No. 5 bars around openings in wall. No. 5 bars shall be developed past the opening but no less than 24 in.
- \( \ell_d \) for a No. 5, top bar, with 1 in. cover per the Lap Splice and Embedment Length Table is 1 ft. 10 in. Thus, use 24 in.

Reinforcement in accordance with 14.3.7 was provided around all openings in the walls.

See Typical Wall Opening in Shearwall.
### Member Design and Detailing – Mat Foundation – Flexure Design

<table>
<thead>
<tr>
<th>NO.</th>
<th>Step</th>
<th>References and Commentary</th>
</tr>
</thead>
</table>
| 1   | Section properties:  
- Plan dimensions: 78.33 ft x 269.5 ft  
- Thickness: $h = 48$ in, uniform, except as required around elevator core  
- $f_c = 4000$ psi  
- Design strip: $b = 13$ ft 7 in.  
- Clear cover = 3 in.  
- $d = 48 - 3 - 1 = 44$ in. | Determine the negative reinforcement at column B.2-22 in the N-S direction.  
The computer program computed required flexural reinforcement for each user-defined strip in both directions in accordance with the design assumptions and general requirements of 10.2 and 10.3, based on critical bending moments (15.2.1 and 15.10.1). |
| 2   | The software indicates that the maximum computed negative moment for the design strip was:  
$M_{nb} = 3,842$ ft-kips  
The critical section for the negative reinforcement was at the face of the column (B.2-22). | The controlling load combination was Eq. 9-2 (9.2.1).  
$1.2D + 1.6L$. |
| 3   | Sections were designed assuming the strength reduction factor $\varphi = 0.9$ for tension-controlled sections, that is, assuming the strain $\varepsilon_t$ in the tensile reinforcement is greater than or equal to 0.005. This assumption was later verified. | Strength reduction factor—9.3.2.1  
Strain limit—10.3.4 |
|     | Assuming a tension-controlled section ($\varphi = 0.9$), the required area of steel was $A_s = 20.1$ in.$^2$ | Procedures to design reinforcement for a rectangular cross-section with tension reinforcement only are covered in textbooks and other technical references (10.2 and 10.3). |
|     | $A_{s,min} = 0.0018bh = 0.0018 \times 163 \times 48 = 14.1$ in.$^2$ | The area of steel provided at each critical section is at least equal to the minimum value required by 10.5.4, which is applicable to mat foundations (R10.5.4). |
| 4   | No. 9 bars @ 12 in. are provided each way at the top and bottom of the mat. In the 13 ft - 7 in. wide section, 13.0 in.$^2$ of reinforcement is provided by the No. 9 bars. | (see Note 2 on Sheet S1.01) |
An additional $20.1 - 13.0 = 7.1 \text{ in.}^2$ is required to satisfy flexural requirements.

Provide 12-No. 7 bars ($A_s = 7.2 \text{ in.}^2$) in the strip in addition to the No. 9 bars @ 12 in.

Total area of steel provided = $13.0 + 7.2 = 20.2 \text{ in.}^2 > 20.1 \text{ in.}^2$

OK.

Verify the section is tension-controlled:

$$a = \frac{A_s f_y}{0.85 f_{c,b}} = \frac{20.2 \times 60}{0.85 \times 4 \times 160} = 2.2 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{2.2}{0.85} = 2.6 \text{ in.}$$

$$\epsilon_1 = \left(\frac{0.003}{c}\right) d - 0.003$$

$$= \left(\frac{0.003}{2.6}\right) \times 44 - 0.003$$

$$= 0.0478 > 0.0050$$

Thus, the section was tension-controlled.

The computed reinforcement tension strain (0.0478) at nominal strength exceeds the minimum strain (0.0050) required to consider the section to be tension-controlled (10.3.4).

Section behavior—10.2.2; 10.2.3; and 10.2.7.

Reference:

Use the No. 9 @ 12 in. grid plus 12-No. 7 bars in the strip.

These bars are shown on Sheet 51.01.
### Member Design and Detailing – Mat Foundation – Shear Design

<table>
<thead>
<tr>
<th>STEP</th>
<th>Structural Design</th>
<th>References and Commentary</th>
</tr>
</thead>
</table>
| 1    | Mat section properties:  
- Thickness: \( h = 48 \) in.  
- \( f_c = 4000 \) psi  
- Design strip: \( b = 13 \) ft 7 in.  
- Clear cover: \( = 3 \) in.  
- \( d = 48 - 3 - 1 = 44 \) in.  
Column section properties:  
- Plan dimensions: \( 36 \) in. \( \times \) \( 30 \) in. | Check two-way shear requirements in the mat foundation at a typical interior column.  
Two-way mat shear requirements needed to be checked at the interior column (30 in. \( \times \) 36 in.). In addition to direct shear, the section needs to resist the fraction of unbalanced moment transferred by eccentricity of shear in the N-S direction. |
| 2    | The following values were obtained from the computer program:  
Soil pressure:  
Service load: \( q_D = 2.8 \) ksf and \( q_L = 0.6 \) ksf  
Factored load: \( q_u = (1.2 \times 2.8) + (1.6 \times 0.6) = 4.3 \) ksf  
The maximum unbalanced moment in the N-S direction was: \( M_u = 166 \) ft-kips | The controlling load combination was Eq. 9-2 (9.2.1)  
\( 1.2D + 1.6L \)  
It is assumed that within the area tributary to an interior column, the soil pressure is constant. The pressures reported above are the average pressures within this area. |
| 3    | First, calculate the factored shear force, \( V_u \), at the critical section:  
\( q_u = (1.2 \times 2.8) + (1.6 \times 0.6) = 4.3 \) ksf  
The factored shear force, \( V_u \), at the critical section was determined:  
\( b_1 = 36 + 44 = 80 \) in.  
\( b_2 = 30 + 44 = 74 \) in.  
\( V_u = 4.3 \left( \frac{300 - \frac{30 \times 74}{144}}{144} \right) = 1,113 \) kips  
where, 300 ft\(^2\) is the approximate tributary area to this column. | The critical section for two-way shear is located at a distance of \( d/2 \) from the face of the supported member (11.12.1.2). Total factored shear stress at a column/wall-mat connection is computed in accordance with 11.12.6 - Transfer of moment in slab-column connections. Two-way shear requirements are satisfied at all locations. |
|      | The maximum factored total shear stress at the critical section was determined: | Transfer of moment - eccentric shear—11.12.6.1, 11.12.6.2, and 13.5.3.1 |
\[ v_u = \frac{V_u}{A_\chi} + \frac{V_v M_v C_{AB}}{J_\chi} \]

\[ A_\chi = 2(b_1 + b_2)1 = 2(60 + 74) \times 44 = 13,552 \text{ in.}^2 \]

\[ J_\chi = \frac{b_1 d^2(1 + b_2)}{3} + d^3 = 382,741 \text{ in.}^3 \]

Equations to determine properties of critical sections for interior, edge, and corner columns are given in textbooks and other technical references \((R11.12.6.2)\).

\[ V_v = 1 - \sqrt{1 - \frac{1}{1 + (2/3)\sqrt{b_1/b_2}}} = 0.41 \]

Factor: unbalanced moment transferred by eccentricity of shear—Eq. 11-39 \((11.12.6.1)\).

Thus, total factored combined shear stress is:

\[ V_u = 1.11 \times 12,000 + 0.41 \times 166.1 \times 12,000 \]

\[ = 82 + 2 = 84 \text{ psi} \]

Nominal shear strength, \( v_c \), is the smallest of:

\[ \left( 2 + \frac{4}{\beta} \right) \sqrt{f_c} = \left( 2 + \frac{4}{1.2} \right) \sqrt{4,000} = 337 \text{ psi} \]

\[ v_c = \left( \frac{4d + 4b}{b_0 + 2} \right) \sqrt{f_c} = \left[ \frac{40 \times 44}{(280 + 74)} + 2 \right] \sqrt{4,000} = 561 \text{ psi} \]

\[ \sqrt[4]{f_e} = 4\sqrt[4]{4,000} = 253 \text{ psi} \]

Two-way action for slabs—\( 11.12.2 \) and \( 11.12.2.1 \) (Eqs. 11-33 to 11-35)

REFERENCE:

MORE REFERENCES

Strength reduction factor—\( 9.3.2.3 \)

\[ \varphi v_c = 0.75 \times 253 = 190 \text{ psi} > 84 \text{ psi} \quad \text{OK.} \]

Thus, two-way shear limits are satisfied for the mat.
**Member Design and Detailing – Mat Foundation – Detailing**

<table>
<thead>
<tr>
<th>Structural Design</th>
<th>References and Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FLEXURAL REINFORCEMENT</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Reinforcement at B.2-22 in the N-S Direction:</strong></td>
<td>It was decided to use continuous uniformly spaced No. 9 @ 12 in. at the top and bottom of the mat in both directions. This layout simplified placement of the bars in the field. Based on the contours of the negative and positive moments, additional top and bottom bars were added for strength where required, and these bars are indicated on the foundation plan.</td>
</tr>
<tr>
<td>- Use the No. 9 @ 12 in. grid plus 12-No. 7 bars in the strip.</td>
<td>The flexural reinforcement in the mat was developed and spiced in accordance with the provisions of 12.14, 12.15, 11.12, and 3.14.</td>
</tr>
</tbody>
</table>

The lengths of the No. 7 bars complied with the provisions of Chapter 12, based on the moment contours from the computer program (see flat slab for similar example).

---

**See Sheet 51.01 for flexural reinforcement on final plan.**

**SHALLOW REINFORCEMENT**

<table>
<thead>
<tr>
<th></th>
<th>Minimum shear reinforcement — 11.5.6.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear reinforcement is not required.</td>
<td></td>
</tr>
</tbody>
</table>

---

![Diagram](image-url)
### 16 Story Hotel / Flat Slab / Shear Wall / Mat Foundation

#### Miscellaneous Design and Detailing – Column to Footing Connection

<table>
<thead>
<tr>
<th>NO.</th>
<th>Structural Design</th>
<th>References and Commentary</th>
</tr>
</thead>
</table>
| 1   | Section properties:  
- Column:  
  - Plan dimensions: 36 in. x 30 in.  
  - Vertical reinforcement: 12-No. 9  
  - $f'_c = 7000$ psi  
  - Clear cover: 1.5 in.  
- Mat Foundation:  
  - Mat dimensions: 78.33 ft x 269.5 ft  
  - $f'_c = 4000$ psi  
  - Clear cover: 3.0 in. | The computer program checked the vertical and horizontal transfer of force at the base of all the columns and walls. Calculations shown here were for a typical interior column. |
| 2   | The maximum factored load and shear from the computer analysis were:  
- $P_u = 2,415$ kips  
- $V_u = 4.8$ kips | The controlling load combination was Eq. 9-2 \( f_{2.1} \)  
\[ 1.2D + 1.6L \]  
Because the interior columns were not designated as part of the lateral-force-resisting system, the shear force was due to gravity effects only. |
| 3   | The bearing strength of the mat under the column footprint was determined:  
\[ q_B = \frac{0.55 f'_{c} A_1}{A_2/A_1} \]  
\[ = 0.65 \left( \frac{0.65 \times 4 \times 36 \times 30}{} \right) \]  
\[ = 2770 \text{ kips} \]  
Therefore, no dowels are needed for strength. | Factored forces and moments at the base of the columns and walls are transferred to the mat foundation by bearing on concrete and by reinforcement (dowels) in accordance with §15.8.1.  
Bearing strength—10.17.1  
Strength reduction factor—9.3.2.4 |
| 4   | Compute the required dowel bars between column and mat.  
\[ A_s,\text{min} = 0.005A_g = 0.005 \times 30 \times 36 = 5.40 \text{ in.}^2 \]  
Use 12-No. 9 dowel bars ($A_g = 12.0 \text{ in.}^2$), which match the | Even though bearing strength of the column and mat concrete is adequate to transfer the factored loads, a minimum area of reinforcement (dowels) is required to transfer force across the interface (§15.8.2). The area of provided dowel reinforcement is at least equal to the minimum value required by |
longitudinal reinforcement in the column.

Development of dowel reinforcement in compression at the column:

\[
\ell_{dc} = \frac{0.02f_yd_b}{\sqrt{\frac{288}{7,000}}} = 0.02 \times 60,000 \times 1.128 = 16.2 \text{ in}
\]

\[
\ell_{dc} = \frac{0.0003f_yd_b}{\sqrt{\frac{288}{7,000}}} = 0.0003 \times 60,000 \times 1.128 = 20.3 \text{ in}
\]

A minimum 20.3 in. dowel length was required into the column.

An embedment of 30\(d_b\) was detailed.

Development of dowel reinforcement in compression at the mat foundation:

\[
\ell_{dc} = \frac{0.02f_yd_b}{\sqrt{\frac{288}{4,000}}} = 0.02 \times 60,000 \times 1.128 = 21.4 \text{ in}
\]

\[
\ell_{dc} = \frac{0.0003f_yd_b}{\sqrt{\frac{288}{4,000}}} = 0.0003 \times 60,000 \times 1.128 = 20.3 \text{ in}
\]

Available length for development into mat = 48 - 3 - (2 x 1.0) - 1.128 = 41.9 in. > 21.4 in.

The dowel bars were extended to just above the longitudinal reinforcement in the bottom of the mat, with standard 90-degree hooks at the ends for ease of construction.

The maximum shear transfer permitted was determined:

\[
V_b = \frac{0.6d_b f_y A_e}{800} = 0.6 \times 60,000 \times 1.128 = 41.9 \text{ kips}
\]

\[
V_u = 4.8 \text{ kips} < 864 \text{ kips}; \text{ thus, shear transfer is permitted at the base of the column.}
\]

Because the shear force is relatively small, it is clear by inspection that the 12-No. 9 dowel bars are adequate as shear-friction reinforcement.

The final detail is shown on sheet 52.01.

The horizontal force transferred to the mat at the base of the column is trivial, but are shown for demonstration purposes.

Maximum shear—11.7.5

Shear-friction provisions of 11.7 were used to check for transfer of lateral forces between the supported members and the mat (15.8.1.4).
## Structural Detailing

Assumptions for the "Lap Splice and Embedment Length" table:
- **General:**
  - Values in table are based on $f_c = 4000$ psi
  - $f_y = 60,000$ psi
  - All tension splices were Class B.
  - No compression splices.
  - No bundled bars
- **For Columns and Beams:**
  - The following equations were used to determine $\ell_d$.
    - For bars No. 6 and smaller:
      $$ \ell_d = \left( \frac{f_y \psi_4 \psi_5 \psi_9}{25 \sqrt{f_y}} \right) d_b $$
    - For bars No. 7 and larger:
      $$ \ell_d = \left( \frac{f_y \psi_4 \psi_5 \psi_9}{20 \sqrt{f_y}} \right) d_b $$
  - Minimum $\ell_d = 12$ in.

## References and Commentary

Per provision 1.2.1(1), the drawings, details, or specifications shall show "anchorage length of reinforcement and location and length of lap splices (1.2.1)." General details for building members were provided that show splice location and minimum splice length and anchorage requirements. These general details do not attempt to show the actual splice and embedment lengths for the various bar sizes. This was accomplished through use of a table, see below.

The development length equations are from 12.2.2 (12.2.1). The use of these equations require that:
- The clear spacing of bars $\geq d_b$
- The clear cover of bars $\geq d_b$
- That stirrups and ties along $\ell_d$ meet the code minimum

These requirements have been met as follows:
- Minimum clear spacing requirements per 7.6.1 shall be $d_b$ and not less than 1 in.
- Minimum clear spacing for columns per 7.6.3 shall be $1.5d_b$ and not less than 1.5 in.
- Clear cover for columns and beams per 7.7.1 is 1.5 in., which is greater than $d_b$ of a No. 11 bar.
- The minimum stirrups and ties were used for all beams and columns for this project.

Therefore, the conditions to use the equations for development length were met. The minimum embedment length is 12 in., per 12.2.1.

The development length equation is from 12.2.2 (12.2.1). This equation allows the depth of cover to be considered in the calculation of the development length. Note that $K_f$ may be conservatively assumed to be zero; ignore transverse reinforcement.
Embodiment and Splice Length Calculation for Columns and Beams

1. Determine the development length for a No. 7 bar. Properties:
   - \( f'_c = 4000 \text{ psi} \)
   - \( f_y = 60,000 \text{ psi} \)
   - \( d_b = 0.875 \text{ in.} \)
   - \( \psi_t = 1.0 \text{ (not a top bar)} \)
   - \( \psi_e = 1.0 \text{ (not epoxi-coated)} \)
   - \( \lambda = 1.0 \text{ (normal weight concrete)} \)

   \[ \xi_d = \left( \frac{3}{4} \frac{f_y}{\sqrt{f'_c} \left( \frac{c_b + K_{tr}}{d_b} \right)} \right) \left( \frac{w_1 w_2 w_3 \lambda}{c_b + K_{tr}} \right) d_b \]

   Development length—12.2.2

2. Determine development length for a No. 7 bar defined as a top bar:
   \( \xi_d = 41.5 \times 1.3 = 54.0 \text{ in. or } 4'6" \)

3. Determine splice length for a No. 7 bar:
   \( \xi_d \times 1.3 = 41.5 \times 1.3 = 54.0 \text{ in. or } 4'6" \)

4. Determine splice length for a No. 7 bar defined as a top bar:
   \( \xi_d \times 1.3 = 54.0 \times 1.3 = 70.2 \text{ in. or } 5'10" \)

Embodiment and Splice Length Calculation for Walls and Slabs

Determine the development length for a No. 7 bar with 1.00 in. cover. Properties (see above):

- \( c_b = 1.00 + 0.5 \times 0.875 = 1.44 \text{ in.} \)
• $K_{tr} = 0$
• $\psi_s = 1.0$ (No. 7 bars or larger)

Modification factor; reinforcement size—12.2.4

Development length—12.2.3

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>$\ell_d = \left( \frac{3}{40} \right) \frac{f_y \psi_s \psi_e \psi_n \lambda}{\sqrt{f' c} \left( \frac{c - K_{tr}}{d_b} \right)} d_b$</td>
<td>$\ell_d = \left( \frac{3}{40} \right) \frac{60,000 \times 1.0 \times 1.0 \times 1.0 \times 1.0}{\sqrt{4000} \left( \frac{1.44 + 0}{0.875} \right)} \times 0.875 = 37.8$ in. or $3^{1/2}$</td>
</tr>
<tr>
<td>6</td>
<td>Determine development length for No. 7 bar defined as a top bar: $\ell_d = 37.8 \times 1.3 = 49.1$ in. or $4^{1/2}$</td>
<td>Modification factor; reinforcement location—12.2.4</td>
</tr>
<tr>
<td>7</td>
<td>Determine splice length for a No. 7 bar: $\ell_d \times 1.3 = 37.8 \times 1.3 = 49.1$ in. or $4^{1/2}$</td>
<td>As stated above, all splices shall be Class B splices as defined in 12.15.1.</td>
</tr>
<tr>
<td>8</td>
<td>Determine splice length for No. 7 bar defined as a top bar: $\ell_d \times 1.3 = 49.1 \times 1.3 = 63.8$ in. or $5^{1/2}$</td>
<td>Modification factor; reinforcement location—12.2.4</td>
</tr>
</tbody>
</table>