Chapter 4 Ductile Coupled Reinforced Concrete Shear Walls

2020 NEHRP Provisions Training Materials
S. K. Ghosh and Prabuddha Dasgupta, S. K. Ghosh Associates LLC

Coupled Walls
Coupled Walls

Courtesy: Cary Kopczynski & Company, Bellevue, WA

Coupled Walls

Courtesy: Magnusson Klemencic Associates, Seattle, WA
Coupled Walls

Courtesy: Cary Kopczynski & Company, Bellevue, WA
Coupled Walls

Coupled shear wall systems are recognized as distinct from isolated shear wall systems in Canadian and New Zealand codes; they are also accorded higher response modification factors in view of their superior seismic performance. ASCE/SEI 7, prior to its 2022 edition, made no such distinction.

Ductile Coupled Shear Walls

Bertero wrote in 1977: “Use of coupled walls in seismic-resistant design seems to have great potential. To realize this potential it would be necessary to prove that it is possible to design and construct “ductile coupling girders” and “ductile walls” that can SUPPLY the required strength, stiffness, and stability and dissipate significant amounts of energy through stable hysteretic behavior of their critical regions.”

Thus, discussion needs to focus not on just coupled walls, but ductile coupled walls consisting of ductile shear walls and ductile coupling beams.
MKA Study:
Non-linear response history analyses were conducted using spectrally matched ground motion records on a variety of coupled shear wall archetypes. Archetypes ranged from 5 to 50 stories in height and contained a range of longitudinal reinforcement ratios in the coupling beams as well as the shear walls.
ACI 318-19 18.10.9 Ductile Coupled Walls

18.10.9.1 Ductile coupled walls shall satisfy the requirements of this section.
18.10.9.2 Individual walls shall satisfy $h_{wcs}/ℓ_w ≥ 2$ and the applicable provisions of 18.10 for special structural walls.
18.10.9.3 Coupling beams shall satisfy 18.10.7 [Coupling beams] and (a) through (c) in the direction considered.
(a) Coupling beams shall have $ℓ_n/h ≥ 2$ at all levels of the building.
(b) All coupling beams at a floor level shall have $ℓ_n/h ≤ 5$ in at least 90 percent of the levels of the building.
(c) The requirements of 18.10.2.5 shall be satisfied at both ends of coupling beams [reinforcement developed for $1.25f_y$].

Special Shear Walls

(a) Wall with $h_w/f_y ≥ 2.0$ and a single critical section controlled by flexure and axial load designed using 18.10.6.2, 18.10.6.4, and 18.10.6.5
Ductile Coupling Beams

- Aspect ratio \( \frac{l}{h} \geq 4 \)
  - Satisfy requirements of 18.6
- Aspect ratio \( \frac{l}{h} < 4 \)
  - Permitted to be reinforced with two intersecting groups of diagonal bars
- Aspect ratio \( \frac{l}{h} < 2 \) and \( V_u > 4\sqrt{f'_c A_{cw}} \)
  - Must be reinforced with two intersecting groups of diagonal bars

Source: http://nees.seas.ucla.edu/pankow
Ductile Coupling Beams

ACI 318-08 Alternate Detail

Confinement of Entire Cross-Section

Source: http://nees.seas.ucla.edu/pankow

2020 NEHRP Provisions

NEHRP Recommended Seismic Provisions for New Buildings and Other Structures
Volume 1: Part 1 Provisions, Part 2 Commentary
FEMA P-800-1/1 September 2020

Source: http://nees.seas.ucla.edu/pankow
2020 NEHRP Provisions

- Part 1: Modifications to ASCE/SEI 7-16
- Part 2: Commentary to the Modifications
- Part 3: Resource Papers

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P695 Study

UCLA Structural/Earthquake Engineering Research Laboratory

Debris Resilience Commonly Used Walls

FEMA P695 Study

Final Report

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Project Sponsors

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Forty-one ductile coupled shear wall buildings were designed using a range of variables expected to influence the collapse margin ratio; the primary variables were building height (i.e., 6, 8, 12, 18, 24, and 30 stories), wall cross section (i.e., planar and flanged walls), coupling beam aspect ratio \( \ell_h/h \) ranging from 2.0 to 5.0, and coupling beam reinforcement arrangement (i.e., diagonally and conventionally reinforced).
The designs were for Risk Category II structures with an importance factor $I_e = 1.0$. It incorporated provisions of ASCE/SEI 7-16 and ACI 318-19 as well as the seismic design parameters specified in FEMA P695 (importance factor, redundancy factor, and site class and spectral values). The redundancy factor $\rho$ was taken equal to 1.0. The seismic spectral acceleration values used are summarized below for seismic hazard $D_{max}$ as specified in FEMA P695.

- $S_S = 1.5g$  \hspace{0.5cm} $F_a = 1.0$  \hspace{0.5cm} $S_{DS} = 1.00 \text{ g}$
- $S_1 = 0.6g$  \hspace{0.5cm} $F_v = 1.5$  \hspace{0.5cm} $S_{D1} = 0.60g$
Additional ACI 318-19 Changes in Special Shear Wall Design

There have been four significant ACI 318-19 code changes, all adopted in our FEMA P695 study, to address the flexural-compression wall failure issue.

(1) **18.10.3.1** (shear amplification) - would typically require design shear (required shear strength) \( V_u \) to be amplified by a factor of up to 3 (similar to New Zealand, Canada).

(2) **18.10.6.4** - requires improved wall boundary and wall web detailing, i.e., overlapping hoops if the boundary zone dimensions exceed 2:1, crossties with 135-135 hooks on both ends, and 135-135 crossties on web vertical bars.

(3) **18.10.6.2(b)** (Wall drift or deformation capacity check) - requires a low probability of lateral strength loss at MCE level hazard (you can think of it as requiring a minimum wall compression zone thickness), and
Additional ACI 318-19 Changes in Special Shear Wall Design

(4) **18.10.2.4** - Minimum wall boundary longitudinal reinforcement, to limit the potential of brittle tension failures for walls that are lightly-reinforced.

Shear Amplification: Concrete Shear Walls

**18.10.3.1** The design shear force \( V_e \) shall be calculated by:

\[
V_e = \Omega_v \omega_v V_u \leq 3V_u \quad (18.10.3.1)
\]

where \( V_u, \Omega_v, \) and \( \omega_v \) are defined in 18.10.3.1.1, 18.10.3.1.2, and 18.10.3.1.3, respectively.

**18.10.3.1.1** \( V_u \) is the shear force obtained from code lateral load analysis with factored load combinations.
Shear Amplification: Concrete Shear Walls

18.10.3.1.2 Ωv shall be in accordance with Table 18.10.3.1.2.

Table 18.10.3.1.2—Overstrength factor Ωv at critical section

<table>
<thead>
<tr>
<th>Condition</th>
<th>Ωv</th>
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<tr>
<td>$h_{wcd}/b_w &gt; 1.5$</td>
<td>Greater of</td>
</tr>
<tr>
<td></td>
<td>$M_p/M_a^{[1]}$</td>
</tr>
<tr>
<td></td>
<td>1.5$^{[2]}$</td>
</tr>
<tr>
<td>$h_{wcd}/b_w \leq 1.5$</td>
<td>1.0</td>
</tr>
</tbody>
</table>

$^{[1]}$ For the load combination producing the largest value of $\Omega_v$.

$^{[2]}$ Unless a more detailed analysis demonstrated a smaller value, but not less than 1.0.

Shear Amplification: Concrete Shear Walls

18.10.3.1.3 For walls with $h_{wcd}/f_w < 2.0$, $\omega_v$ shall be taken as 1.0. Otherwise, $\omega_v$ shall be calculated as:

$$
\omega_v = 0.9 + \frac{n_z}{10} \quad \text{if } n_z \leq 6
$$

$$
\omega_v = 1.3 + \frac{n_z}{30} \quad \text{if } n_z > 6
$$

(18.10.3.1.3)

where $n_z$ shall not be taken less than the quantity $0.007h_{wcd}$. 
Earthquake Force-Resisting Structural Systems of Concrete — ASCE/SEI 7-22

<table>
<thead>
<tr>
<th>Basic Seismic Force-resisting System</th>
<th>Detailing Reference Section</th>
<th>R</th>
<th>Ω₀</th>
<th>C₆</th>
<th>System Limitations And Building Height Limitations (Ft) By Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
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</tr>
<tr>
<td>A. Bearing Wall System</td>
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<tr>
<td>1. Special reinforced concrete shear walls</td>
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<td>2₁/₂</td>
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<td>2. Ductile Coupled reinforced concrete shear walls&lt;sup&gt;q&lt;/sup&gt;</td>
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<td>NL</td>
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<tr>
<td>3. Ordinary reinforced concrete shear walls</td>
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<td>2₁/₂</td>
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<td>NL</td>
</tr>
</tbody>
</table>

<sup>q</sup> Structural height, \(h_{\text{d}}\), shall not be less than 60 ft (18.3 m).

Minimum height is intended to ensure adequate degree of coupling and significant energy dissipation provided by the coupling beams.

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<table>
<thead>
<tr>
<th>Basic Seismic Force-resisting System</th>
<th>Detailing Reference Section</th>
<th>R</th>
<th>Ω₀</th>
<th>C₆</th>
<th>System Limitations And Building Height Limitations (Ft) By Seismic Design Category</th>
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<td>4₂/₃</td>
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</table>

<sup>q</sup> Structural height, \(h_{\text{d}}\), shall not be less than 60 ft (18.3 m).

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Earthquake Force-Resisting Structural Systems of Concrete — ASCE/SEI 7-22

| Basic Seismic Force-resisting System | Detailing Reference Section | \( R \) | \( \Omega_0 \) | \( C_d \) | System Limitations And Building Height Limitations (Ft) By Seismic Design Category |
|-------------------------------------|-----------------------------|------|------|------|---------------------------------|--------|--------|--------|--------|
|                                     |                             |      |      |      | B | C | D | E | F |
| D. Dual Systems with Special Moment Frames |                             |      |      |      | 33 |
| 3. Special reinforced concrete shear walls | 14.2 | 7 | 2\( \frac{1}{2} \) | 5\( \frac{1}{2} \) | NL | NL | NL | NL | NL |
| 4. Ductile Coupled reinforced concrete shear walls | 14.2 | 8 | 2\( \frac{1}{2} \) | 8 | NL | NL | NL | NL | NL |
| 5. Ordinary reinforced concrete shear walls | 14.2 | 6 | 2\( \frac{1}{2} \) | 5 | NL | NL | NP | NP | NP |

\( ^a \) Structural height, \( h_d \), shall not be less than 60 ft (18.3 m).

FEMA Publication

2020 NEHRP Recommended Seismic Provisions: Design Examples, Training Materials, and Design Flow Charts

FEMA P-2192-V1/November 2021
Volume I: Design Examples
Example Problem
Design of a Special Reinforced Concrete Ductile Coupled Wall

Introduction
A 22-story reinforced concrete residential building is designed following the requirements of ASCE/SEI 7-22, and ACI 318-19. The building consists of a flat plate-column gravity system with a central core, formed by four reinforced concrete coupled structural walls, which acts as the seismic force-resisting system. The structural walls are designed as Ductile Coupled Reinforced Concrete Shear (Structural) Walls.

A computer rendering of the building framing is shown on the next two slides. The plan view of the building changes from one floor to another. A plan view of the second floor of the building is shown.
Example Building Configuration

3D View

Example Building Configuration

Second Floor Plan View
Design Criteria

- Member Sizes:
  - Shear walls: 26 in. thick
  - Slabs (2nd and 3rd floors): 8 in. thick
    (4th floor and higher): 7.5 in. thick
  - Gravity columns: Various sizes

- Material properties:
  - Concrete (used in structural walls and columns): $f'_c = 8000$ psi (all floors)
  - Concrete (used in slabs): $f'_c = 6000$ psi (floors)
  - All members are constructed of normal weight concrete ($w_c = 150$ pcf)
  - Reinforcement (used in all structural members): $f_y = 60,000$ psi

- Service Loads:
  - Superimposed dead load: 25 psf (includes SDL on the floor plus the weight of cladding distributed over the floor slab.)
  - Live load: Based on the 40 psf live load prescribed in ASCE/SEI 7-22 Table 4.3-1 for residential buildings (private rooms and corridors serving them), a reduced live load of 20 psf is used in the example.
  - Reduced roof Live load: 20 psf

- Seismic Design Data:
  - Risk Category: II
  - Seismic importance factor, $I_e = 1.0$
  - Site Class: D
## Design Criteria

### Seismic Design Data (contd.):

- The maximum considered earthquake spectral response acceleration:
  - At short periods, $S_S = 1.65g$, and
  - At 1-sec period, $S_1 = 0.65g$.

- The maximum considered earthquake spectral response acceleration (site modified):
  - At short periods, $S_{MS} = 1.65g$, and
  - At 1-sec period, $S_{M1} = 0.98g$.

- Design Spectral Response Acceleration Parameters (at 5% damping):
  - At short periods: $S_{DS} = \frac{2}{3} \times \frac{S_{MS}}{g} = \frac{2}{3} \times 1.65 = 1.10$
  - At 1-sec period: $S_{D1} = \frac{2}{3} \times \frac{S_{M1}}{g} = \frac{2}{3} \times 0.98 = 0.65$

### Seismic Design Data (contd.):

- Long-period transition period, $T_L = 8$ sec
- Ductile Coupled Reinforced Concrete Structural Walls ... $R = 8; C_d = 8.0, \Omega_0 = 2.5$ (ASCE/SEI 7-22 Table 12.2-1)
- Seismic Design Category: Based on both $S_{DS}$ (ASCE/SEI 7-22 Table 11.6-1) and $S_{D1}$ (ASCE/SEI 7-22 Table 11.6-2), the Seismic Design Category (SDC) for the example building is D.
Design Procedure

Although ASCE/SEI 7-22 permits the Equivalent Lateral Force procedure to be used in all situations, the modal response spectrum analysis (MRSA) procedure (ASCE/SEI 7-22 Section 12.9.1) is used in this example. However, as part of the MRSA procedure, base shear is also determined using Equivalent Lateral Force (ELF) procedure. This is because ASCE/SEI 7-22 requires that the base shear obtained from MRSA be scaled up to match the ELF base shear.

The building was modeled in ETABS 2016, and the total seismic weight was obtained from the program as \textbf{43,099 kips}.

Modal Response Spectrum Analysis

A 3-D modal response spectrum analysis (MRSA) is performed using ETABS (v2016).
- Semi-rigid diaphragms are assigned at each level.
- The effective cracked member stiffnesses used in the analyses are as follows:
  - Columns and shear walls, $l_{\text{eff}} = 0.7l_g$
  - Coupling beams, $l_{\text{eff}} = 0.25l_g$
  - Gravity columns, $l_{\text{eff}} = 0.1l_g$ (with pinned connections at the base)
- Adequate number of modes are considered in the modal analysis to incorporate \textbf{100\%} of the modal mass in each of x- and y-directions. Also, appropriate scale factors are applied to the base shears calculated in the x- and y-directions to amplify them to those calculated in the ELF procedure.
### Floor Forces from MRSA

<table>
<thead>
<tr>
<th>Story</th>
<th>Elevation (ft)</th>
<th>X-Dir (kip)</th>
<th>Y-Dir (kip)</th>
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### Story Drifts from MRSA (X-Direction)

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<th>$\delta_x$ (in.)</th>
<th>$C_d$</th>
<th>$\delta_x$ (in.)</th>
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### Story Drifts from MRSA (Y-Direction)

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<th>$C_d$</th>
<th>$\delta_x$ (in.)</th>
<th>Relative Drift (%)</th>
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<tr>
<td>L09</td>
<td>10</td>
<td>1.21</td>
<td>8</td>
<td>9.65</td>
<td>1.15</td>
</tr>
<tr>
<td>L08</td>
<td>10</td>
<td>1.03</td>
<td>8</td>
<td>8.27</td>
<td>1.13</td>
</tr>
<tr>
<td>L07</td>
<td>10</td>
<td>0.86</td>
<td>8</td>
<td>6.92</td>
<td>1.09</td>
</tr>
<tr>
<td>L06</td>
<td>10</td>
<td>0.70</td>
<td>8</td>
<td>5.60</td>
<td>1.04</td>
</tr>
<tr>
<td>L05</td>
<td>10</td>
<td>0.54</td>
<td>8</td>
<td>4.35</td>
<td>0.98</td>
</tr>
<tr>
<td>L04</td>
<td>13</td>
<td>0.40</td>
<td>8</td>
<td>3.18</td>
<td>0.87</td>
</tr>
<tr>
<td>L03</td>
<td>15</td>
<td>0.23</td>
<td>8</td>
<td>1.82</td>
<td>0.67</td>
</tr>
<tr>
<td>L02</td>
<td>16.25</td>
<td>0.08</td>
<td>8</td>
<td>0.62</td>
<td>0.32</td>
</tr>
</tbody>
</table>

### Story Drift Limitation

According to ASCE/SEI 7-22 Section 12.12.1, the calculated relative story drift at any story must not exceed 2% (ASCE/SEI 7-22 Table 12.12-1 for all other buildings in Risk Category I and II). As can be seen from the previous slide, this is satisfied in all stories.
Design of Shear Wall

- The design of one of the shear walls at the base of the structure is illustrated in this example in accordance the provisions of ACI 318-19.
- One L-shaped segment of the shear wall core is designed as two flanged walls.
- Orthogonal combination of seismic forces is NOT required as axial loads on the wall from seismic forces are less than 20% of the design axial strength.

Design of Shear Wall – Design Loads

Seismic forces acting along x-axis are considered in this design example. The design calculations for the seismic forces acting along the y-axis are similar and are not shown. However, the final wall configuration will incorporate effects of seismic forces in both directions.

<table>
<thead>
<tr>
<th>Load Combinations</th>
<th>Axial Force, $P_u$ (kips)</th>
<th>Shear Force, $V_u$ (kips)</th>
<th>Bending Moment, $M_u$ (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$1.4D$</td>
<td>6335</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>$1.2D + 1.6L + 0.5L_r$</td>
<td>6071</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>$(1.2+0.2S_{os})D + pQ_e + 0.5L$</td>
<td>10,015</td>
<td>576</td>
</tr>
<tr>
<td>4</td>
<td>$(0.9D - 0.2S_{os}D) + pQ_e$</td>
<td>6460</td>
<td>573</td>
</tr>
<tr>
<td>5</td>
<td>$(0.9D - 0.2S_{os}D) - pQ_e$</td>
<td>-378</td>
<td>573</td>
</tr>
</tbody>
</table>
Design of Shear Wall – Design for Shear

- Height of the shear wall, $h_{wcs} = 2811$ in. (234.25 ft)
- Length of the shear wall, $\ell_w = 164$ in. (13.67 ft)
- $h_{wcs}/\ell_w = 2811/164 = 17.1$

**ACI 318-19 (hereafter ACI 318) Section 18.10.2.2**
At least two curtains of reinforcement shall be used if $V_u > 2A cv\lambda \sqrt{f_c}$ or $h_{wcs}/\ell_w \geq 2.0$. In this case, $h_{wcs}/\ell_w = 17.1 > 2.0$.
So, at least two curtains of reinforcement are required.

Design of Shear Wall – Design for Shear

**ACI 318 Section 18.10.3.1**
Design shear force, $V_e = \Omega v V_u \leq 3V_u$

- For walls with $h_{wcs}/\ell_w > 1.5$, $\Omega v$ is the greater of $M_{pr}/M_u$ and 1.5. The probable moment strength $M_{pr}$ is unknown at this stage. So, it is assumed that $\Omega v = 1.5$. This may very well prove to be unconservative. Once the flexural reinforcement has been provided, this will be verified or corrected, if necessary.
- For walls with $h_{wcs}/\ell_w \geq 2.0$ and the number of stories above critical section, $n_s > 6$,
  $\omega v = 1.3 + n_s/30 \leq 1.8$
  - In this example, $n_s = 22$. $n_s$ cannot be taken less than the quantity $0.007h_{wcs}$ ($= 19.68$), which is satisfied.
  - $\omega v = 1.3 + 22/30 = 2.03 \rightarrow \omega v = 1.8$
Design of Shear Wall – Design for Shear

**ACI 318 Section 18.10.3.1**
Design shear force, \(V_e = \Omega_w \cdot V_u \leq 3V_u\)
- \(V_e = 1.5 \times 1.8 \times 576 = 1555\) kips (governs)
- \(V_e = 3V_u = 3 \times 576 = 1728\) kips

**ACI 318 Section 18.10.4.4.**
The maximum nominal shear strength, \(V_n\), allowed for a wall section is

\[
10A_{cv} \sqrt{f_c} = 10 \times 4264 \times \sqrt{f_c} / 1000 = 3813 \text{ kips}
\]

So, \(\varphi V_n = 0.75 \times 3813 = 2860\) kips > \(V_e\) → The provided wall section size is acceptable.

Design of Shear Wall (Grade 60 Reinforcement)
Design of Shear Wall (Grade 80 Reinforcement)

- Use of Grade 80 steel leads to a considerable reduction in the amount of reinforcement in the wall. In addition to the smaller bar sizes, lesser congestion in the special boundary elements is especially noticeable. However, the vertical spacing of the transverse hoops and cross-ties in the special boundary elements remained 5 in. as that in the Grade 60 design. This is because the maximum value of that spacing is limited to 6 times the diameter of the smallest longitudinal bar. So, smaller bar sizes achieved by higher strength reinforcement ironically led to a tighter spacing compared to what would be necessary for confinement alone. The vertical spacing of the horizontal shear reinforcement is also smaller than what is required for resisting shear so that it matches the spacing of transverse reinforcement in the boundary elements for construction efficiency. Thus, some of the gains achieved by using Grade 80 reinforcement are negated by various other considerations.
A coupling beam oriented along the y-axis of the building at the second floor level is selected for this example. The dimensions of the beam are given below:

- Clear span of the beam, $\ell_n = 76$ in. (6.33 ft)
- Height of the beam, $h = 28$ in. (2.33 ft)
- Width of the beam, $b_w = 26$ in. (2.17 ft)
- $\ell_n/h = 76/28 = 2.7$

Since $2 < \ell_n/h < 4$, per ACI 318 Section 18.10.7.3, this beam can be designed as a deep coupling beam using two intersecting groups of diagonally placed bars, or as a special moment frame flexural member in accordance with the ACI 318 Sections 18.6.3 through 18.6.5. The second option is adopted for this example.

### Design of Coupling Beam – Design for Shear

- #8 long. top reinf.
- Six-legged #4 stirrup @ 6 in.
- #8 long. bottom reinf.
Questions