Neil Hawkins, ACI Honorary Member and ASCE Distinguished Member, is Professor Emeritus of the University of Illinois. He was a Ph.D. student at Illinois at the same time as Dr. Corley and worked with Dr. Corley as a Research Engineer investigating flat plate construction at the Portland Cement Association in 1966-67.

FLAT PLATE AND FLAT SLAB CONSTRUCTION
Neil M. Hawkins, Professor Emeritus, University of Illinois

A Tribute to the Lasting Contributions and Legacy of Our Friend and Colleague Dr. W Gene Corley
ACI Convention, Phoenix, AZ, Sunday October 20, 2013

DISCUSSION TOPICS
Gene’s Early Professional Years

- Equivalent Frame Analysis
  - SRS 218 Univ. of Illinois – Ph.D. Thesis – June 1961
  - Concrete International – Dec. 1983 – w. Dan Vanderbilt
- Testing and Analysis of Flat Plate and Flat Slab System Shear Strengths
  - ACI SP-30 –1971– Moment and Shear Transfer to Columns – w. NMH
  - ACI SP-42 – 1974: Moment Transfer with Shearheads – w. NMH
  - WCEE 1973: Ductile Flat-Plate Structures to Resist EQ – w. JEC & PHK
  - ACI SP-59: 1979: Shear in Two-Way Slabs – ACI Approach
1971 AND 1941 DEFORMATION ASSUMPTIONS

1971 SLAB STIFFNESS ASSUMPTIONS

TORSIONAL MEMBER STIFFNESS ASSUMPTIONS

Where \( C = \text{Torsional Constant} \)

In Corley's thesis the unit twisting moment, Fig 3(B), was uniform over the length \( L_2 \). Jirsa modified Corley's distribution to that shown based on pattern loading considerations

EQUIVALENT COLUMN STIFFNESS

For moment distribution procedures the equivalent column stiffness \( K_{ec} \) was defined by:

\[
\frac{1}{K_{ec}} = \frac{1}{K_c} + \frac{1}{K_t}
\]

Where \( K_c = \text{column flexural stiffness} \)

1871 SLAB STIFFNESS ASSUMPTIONS

Fig 17. Simplified physical models illustrating the intent of Section 13.7.4

EQUIVALENT COLUMN STIFFNESS

For moment distribution procedures the equivalent column stiffness \( K_{ec} \) was defined by:

\[
\frac{1}{K_{ec}} = \frac{1}{K_c} + \frac{1}{K_t}
\]

Where \( K_c = \text{column flexural stiffness} \)

K_t = torsional stiffness of members framing into column

LAYOUT OF 9 PANEL U of 1 1/4 SCALE MODEL

COMPARISON OF MEASURED AND COMPUTED SERVICE LOAD MOMENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Moment Coefficients of</th>
<th>Uniform Load</th>
<th>Non-Uniform Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>0.929 0.929 0.929 0.929 0.929 0.929 0.929 0.929 0.929</td>
<td>0.929 0.929 0.929 0.929 0.929 0.929 0.929 0.929 0.929</td>
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<tr>
<td>2</td>
<td>0.85 0.85 0.82 0.79 0.76 0.73 0.70 0.67 0.64</td>
<td>0.85 0.85 0.85 0.85 0.85 0.85 0.85 0.85 0.85</td>
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<tr>
<td>3</td>
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<tr>
<td>4</td>
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</tr>
<tr>
<td>5</td>
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<td>0.64 0.64 0.64 0.64 0.64 0.64 0.64 0.64 0.64</td>
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</table>

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COMPARISON WITH PCA ¾ SCALE FLAT PLATE RESULTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Measured Uniaxial Load</th>
<th>Design Uniaxial Load</th>
<th>Actual Measured Uniaxial Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

EQUIVALENT FRAME PROCEDURE LIMITATIONS

- Method assumes analysis by moment distribution methods
- Method calibrated for gravity loadings only by comparison to U of I ¼ scale and PCA ¾ scale tests
- Method based on stiffness of uncracked sections
- Method not calibrated for lateral loadings but theoretical studies suggest using a cracked section stiffness equal to 1/3rd uncracked section stiffness. See ACI 318R13.5.1.2
- The method is extensively used and remains essentially unchanged since 1971.

PUNCHING SHEAR

- Flat plate for PCA and U of I tests designed for 70 psf LL and 86 psf DL. Grade 40 steel: 3000 psi concrete.
- Both slabs failed by punching at an interior column. Strains in the top steel at the column face ≥ 7 times the yield strain at punching. Failure load of 369 psf and was only 85% of the ACI 4√f’c value.
- Computed yield line strength was 350psf. Based on shape of the load-slab midspan deflection curves and the limited spread of reinforcement yielding across the width of the slab a capacity greater than the 369psf was likely if not for the punching failure.
- Punching was classified as a “secondary” failure due to the extensive yielding of the top reinforcement around the column prior to failure.

PUNCHING SHEAR ISSUES

- How to prevent the “secondary” punching failure and enable large slab deflections before failure? Answer: Shear reinforcement but what type?
- How to evaluate punching strength when there is also moment being transferred from slab to column?
- Under Gene’s leadership PCA set out to make significant contributions to addressing both those issues.

SHEAR REINFORCEMENT STUDIES

- Shearheads
  - 1930 Wheeler Patent Shearhead
  - PCA TEST SPECIMENS
  - 1966 PCA TEST SHEARHEADS

- Shearhead increases shear capacity in the same way as a larger column. For warning of failure shearhead should yield before punching. Then critical section for shear does not extend to end of shearhead
SHEAR REINFORCEMENT STUDIES
Shearhead – Determination of Required Capacity

\[ N_s = \frac{V_s}{b_1} \left[ \theta + K \left( \frac{b_2}{2} - \frac{b_1}{2} \right) \right] \]

SHEAR DETERMINED FROM STRAIN GAGE READINGS

IDEALIZED SHEAR
\[ K = \frac{E_I}{E_I} \text{ SHEARHEAD} \]

EI COMPOSITE SECTION WIDTH \((c + d)\) \[ K \geq 0.15 \]

SHEAR REINFORCEMENT STUDIES
Shearhead – Location of Critical Section for Shear

\[ \text{Model} \]
\[ \text{Torsional, Flexural and Overall Response} \]

SHEAR REINFORCEMENT STUDIES
Shear and Moment Transfer – Existing ACI Code

Fraction \( \gamma_f M_\text{f} \) to be transferred by flexure within lines 1.5h either side of column where

\[ \gamma_f = \frac{1}{1 + 2.3 \cdot b_2 / b_1} \text{ for } \gamma_f \leq 1 \]

and \( b_2 = c_1 + d \)

For RC slabs and exterior columns \( \gamma_f \) can be increased to 1.0 provided \( V_\text{f} \) does not exceed 0.75\( \phi \)\( V_c \) for edge columns and 0.50\( \phi \)\( V_c \) for corner columns. At interior columns \( \gamma_f \) can be increased by 25% but to not greater than 1.0 provided \( V_\text{f} \leq 0.40 \phi \)\( V_c \) and \( a_2 \geq 0.010 \).

Determining Fraction of \( M \) Transferred by Reinforcement

UNDERSTANDING SHEAR AND MOMENT TRANSFER BEAM ANALOGY

UNDERSTANDING SHEAR AND MOMENT TRANSFER BEAM ANALOGY - EXTERIOR COLUMN STRENGTH

VARIABLES:
- Sheahead - Shape, Length, Area
- Column Size - 12 x 8 in, 12 x 12 in
- Grade 60 Steel
- Sanded Lightweight Concrete 3,000 psi
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**SHEAR REINFORCEMENT STUDIES**

**Exterior Column Connections – Test Setup**

**SHEAR REINFORCEMENT STUDIES**

**Exterior Column Connections – Loading Response**

- D = 12 x 8; C = 12 x 12 column
- N = No Shearhead
- C = Channel Sections
- H = I Sections
- Under-reinforced CH4; CC5; DC2
- Projections: 17.5, 21, 21 in
- Over-reinforced CH1,2,3
- Projections: 8.5, 11.5, 14.5 in
- Over-reinforced CT1, CC1, CC2
- Projections: 14.5, 21, 21 in

**SHEAR REINFORCEMENT STUDIES**

**Exterior Column Connections – Critical Sections**

For shear stress $v_1$ due to Shear

For shear stress $v_2$ due to Moment Transfer

For Design $v_1 + v_2 = v_o \leq \phi v_o$

**SHEAR REINFORCEMENT STUDIES**

**Exterior Column Connections – Shearhead Strength Requirements**

Current Code Requirement For Plastic Moment Strength

**WHAT STILL NEEDS TO BE ADDRESSED?**

**Slabs Without Shear Reinforcement – Flexural Strength Limit**

- Recognize Relevance of Muttoni’s Critical Shear Crack (CSC) Theory
- Aggregate Interlock Along CSC Is Lost When There Is General Yielding of Reinforcement in the Vicinity of Column
- Per Ghali, Strength for General Yielding is 8m where m is flexural strength per unit width
- Require $\phi V_{max} \leq \phi V_{flex} = \phi 8m$ – Needed for $\lambda m \rho$

**WHAT STILL NEEDS TO BE ADDRESSED?**

**Slabs Without Shear Reinforcement – Depth Effect**

$k_v = \frac{3}{\sqrt{d}}$
WHAT STILL NEEDS TO BE ADDRESSED?
Slabs With Shear Reinforcement

• Develop Conceptually Consistent Punching Shear, Moment Transfer, and Ductility Provisions For Connections With Shear Reinforcement

Cover Stirrup Reinforcement,
Stud Rail Reinforcement,
Fortress Reinforcement,
Shearhead Reinforcement.