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Advancing concrete knowledge


Errors in the Design and Construction of Concrete Structures

ACI Fall 2010 Convention
October 24 - 28, Pittsburgh, PA

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ACI Web Sessions

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ACI conventions provide a forum for networking, learning the latest in concrete technology and practices, renewing old friendships, and making new ones. At each of ACI's two annual conventions, technical and educational committees meet to develop the standards, reports, and other documents necessary to keep abreast of the ever-changing world of concrete technology.

With over 1,300 delegates attending each convention, there is ample opportunity to meet and talk individually with some of the most prominent persons in the field of concrete technology. For more information about ACI conventions, visit www.aciconvention.org.




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This ACI Web Session includes 2 speakers presenting at the ACI fall convention held in Pittsburgh, PA, October 24 – 28, 2010.

Additional presentations will be made available in future ACI Web Sessions.

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Errors in the Design and Construction of Concrete Structures

ACI Fall 2010 Convention
October 24 - 28, Pittsburgh, PA

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James LaBelle, PE, Doc.E., Senior Associate, Computerized Structural Design. Professional Engineer for over 30 years, Dr. LaBelle has designed structures and components comprised of various materials. He has investigated, and in many cases designed repairs for, various structures including facades, building framing, hollow sidewalks, parking structures, a major scaffold collapse on a high-rise building, and wood pile foundations. His research activities include sandwich beams and connections in aluminum. His publications include the doctoral dissertation and papers on sandwich beam analysis, façade investigations and repairs, and screws installed in aluminum. He is a member of ASCE, ASTM, and AAMA. Other activities include teaching as a guest lecturer for engineering topics and testifying in support of litigation related to investigations.

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Why & What To Do? CRACKED P-T BEAMS & STEEPLY-CRACKED COLUMN

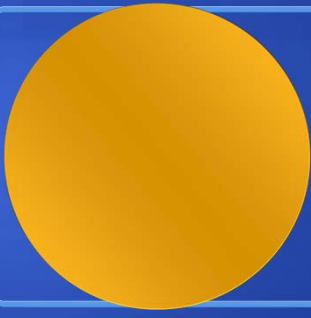
Two Case Studies in
Investigation and Repair

James LaBelle, P.E., Doc.E.
Computerized Structural Design, S.C.

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CRACKED P-T (Post-tensioned) BEAMS

MULTI-STORY
BUILDING;
1980s



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Discovery of P-T Beam Cracks

- Incidental to another activity, ceiling tiles were removed below two post-tensioned beams
- Unexpected cracking across the bottom of the beams was noticed
- Cracking was located at about midspan
- Examination of several other beams and girders, at same level, revealed additional cracking

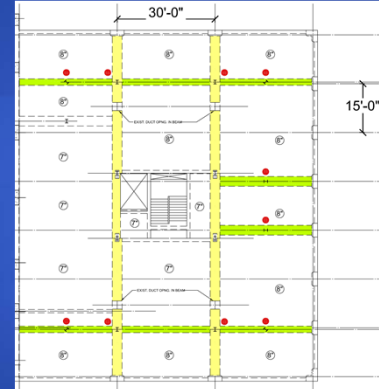
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Background

- Initial plan called for a structure using primarily post-tensioned concrete beams and slabs and reinforced concrete columns
- Late in design phase, a steel-framed two-story structure (library and file storage) was added to building at what had been a set-back roof level
- Some of the new steel columns were supported at midspan of post-tensioned flexural members
- Beam spans were about 30'; girders spanned 45'

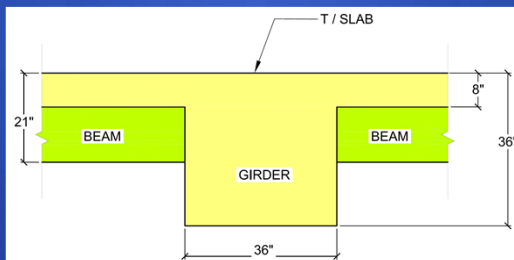
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Partial plan of floor



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Section through girder



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Investigation

- Rest of building included some exposed PT beams. These were essentially free of cracks.
- Owner authorized investigation of affected beams
- Structural drawings, but not original calculations, were available
- Analysis showed that, in the worst case for positive moment, required (factored) moment was about 44% larger than design capacity

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Investigation (cont'd.)

- Calculated negative moment (worst case) was about 130% larger than design capacity
- Further study determined that secondary moments likely had not been included in original design
- After conducting a file-loading survey, an attempt was made to limit amount of file storage
- Difficult to enforce the load limits

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Load Monitoring

- A deflection-monitoring system was designed and installed at two P-T beams with midspan loading
- LVDTs and dial gages were supported by custom steel trusses.
- Deflections were measured to the nearest 0.001"
- A system to acquire and store the LVDT data was installed
- Data could be read remotely

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Monitoring (cont'd.)

- Sensitivity was sufficient to see normal daily changes in deflected shape due to temperature and live-load fluctuations
- At one point in time, average deflection was found to have increased significantly
- A site visit revealed that substantial file-loading had been added

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Strengthening of Structure

- Owner ultimately decided to have this part of building strengthened, rather than lower its allowable loading
- After study of options, CSD chose external post-tensioning (7-wire strands, 0.6" diam., 4 per tendon, 1 tendon each side of beam)
- A V-shaped profile provided an upward force to counteract the concentrated column-loads
- Space below affected part of floor had to be temporarily emptied for construction

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External post-tensioning

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Strengthening (cont'd.)

- Before coring of transverse girders and beams, radar-imaging was employed at proposed hole-locations to determine as-built positions of internal rebar and unbonded tendons
- Custom saddles and other connections were designed and installed to transfer forces from the new external tendons to the existing structure
- Fire protection: steel mesh installed over tendons and fire-resistant material sprayed on mesh

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A Repaired Span

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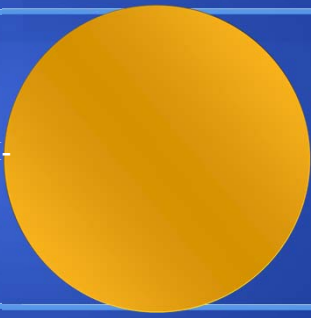
Partial plan of floor

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COLUMN WITH VERY STEEP CRACK

BASEMENT of HIGH-RISE BUILDING

A 1980s building, more than 20 stories high



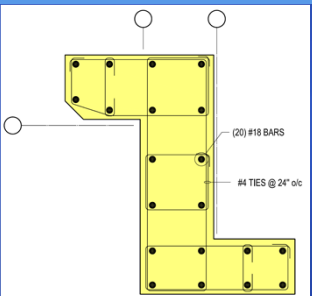
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Initial observations

- Steep crack noticed on exposed side of a major column
- Z-shaped cross-section; 6.5' deep
- Reinforcing: (20) - #18 bars; #4 ties at 24"
- Concrete strength: $f_c' = 5.6$ ksi

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Column section (6.5' deep)




(20) #18 BARS

#4 TIES @ 24" o/c


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Column, as found



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Lower part of crack




diagonal crack

North

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Middle portion of crack



diagonal crack

crack

North

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Close-up of crack



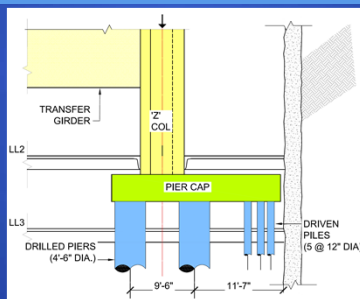
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Calculated capacities

- $\Phi P_n = 10,060$ kips, calculated axial design-strength
- Allowable axial ranged from 5,900 k (if all live load) to 7,200 k (if all dead).
- Allowable for two drilled-piers (used) given as 8,588 kips
- Optional single pier (not used) given as 7,495 kips

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Overview



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Foundation

- Documents showed that piles were added, after original design using only drilled-piers
- Concern about bell strength, as constructed, led to adding 5 driven piles
- All piles located on one side of the drilled piers, apparently to avoid loss of parking space
- As a result, foundation's centroid did not match column's center

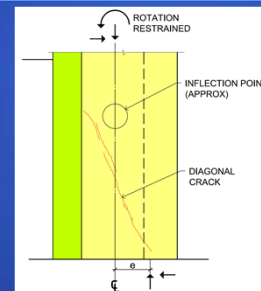
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Foundation (cont'd.)


- This eccentricity caused a moment at the column's base, and a horizontal shear in the column
- The eccentricity was calculated and found to equal about 1.8', based on certain assumptions about behavior of drilled piers and driven piles

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Loaded Column



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Dr. Andrew Scanlon, is an ACI Fellow and a Professor of Civil Engineering at Penn State University. He is currently Chair of ACI Committee 435, Deflections of Concrete Building Structures, and a Member of ACI Committees 224, Cracking and 348, Structural Safety and Reliability. He received his B.S. degree from the University of Glasgow and his Ph.D. from the University of Alberta.

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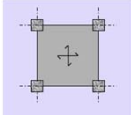
Errors in Design and Construction of Two-way Slab Systems

by
Andrew Scanlon
Penn State University

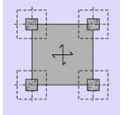
ACI FALL CONVENTION
Pittsburgh, 2010

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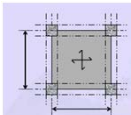
Two-way Slabs



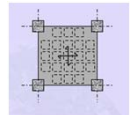
Flat Plate



Flat Slab



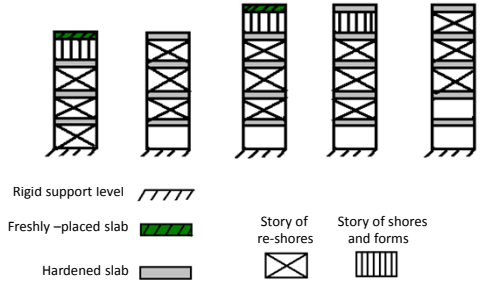
Two-way slab with beams



Waffle Slab

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Shoring and re-Shoring



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Sources of potential problems

- **Design**
 - Flexural capacity
 - Punching shear capacity
 - Deflections
 - Reinforcement detailing-integrity steel

- **Construction**
 - Reinforcement placement
 - Shoring - re-shoring procedures
 - Early-age concrete properties

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Punching Shear Capacity

$$V_u < \phi V_c$$

$$= \phi 4vf'_c b_o d$$

(Delatte)

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$$V_u < \phi V_c = \phi 4vf'_c b_o d$$

- Depends on v'_c measure of diagonal tension strength
- b_o = critical perimeter measured at $d/2$ from column face. Depends on:
 - * column dimensions
 - * effects of slab openings near the column
- d = effective depth of reinforcement
- shear stress due to moment transfer between slab and column

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Case Studies

Punching shear failures reported as early as 1911

2000 Commonwealth Avenue, Boston, 1971
 16 stories, mechanical room on roof. Was started in 1969

- Punching shear failure, main roof
- Collapse of roof slab → progressive slab collapse of the structure

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Collapse occurred while concrete was being placed at the roof level

- Inadequate concrete strength (cold weather)
- Detailing and placement errors in reinforcement
- Inadequate shoring → probably no shoring?
- Inadequate inspection and supervision during construction
- Slab thickness too small for shear
- Heavy construction loads → storage of boiler and construction equipment

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Skyline Plaza-Bailey's Cross-roads 1973

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Skyline Plaza-Bailey's Cross-roads 1973

Progressive collapse from 24th floor

- Central portion of the building
- One floor per week
- Concrete being placed on 24th floor
- Possibly inadequate shoring
- Cold weather? → low strength
- Probably a punching shear failure

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Harbour Cay Condominium Collapse



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Harbour Cay Condominium, Cocoa Beach, Florida 1981

- Under construction, 5 story, roof slab
 - slab thickness low 8" vs 11"
 - top steel too low
- ↳ Combined effect
- Large deflections and severe cracking noted prior to collapse
- No punching shear calculations had been made
- Severe design errors

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Deflections

- Two-way systems

Slabs – relatively large span-depth ratio

$$\frac{l}{h} \sim 30 \text{ to } 36$$

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Deflections

- Damage → economic loss
- Types of damage
 - Perceptible sagging
 - Damage to non-structural elements
 - Impairment of function

Installation of prefabricated elements

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Deflections

Serviceability requirements have to be project specific → difficult to cover all possibilities in code

Table 9.5(C): Minimum thickness of slabs without interior beams §9.5 of ACI 318-08 for two-way construction (reproduced with permission from ACI 318-08)

f_c , psi	Without drop panels [†]			With drop panels [†]		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams		Without edge beams	With edge beams	
40,000	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
60,000	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
75,000	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

[†]For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
[‡]For f_c between the values given in the table, minimum thickness shall be determined by linear interpolation.
[§]Drop panels as defined in 13.2.5.
[¶]Slabs with beams between columns along exterior edges. The value of c_{ep} for the edge beam shall not be less than 0.8.

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Deflection Control provisions

- Minimum thickness (span-to-depth ratio)
- Deflection Calculation
- Construction phase

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No calculations made to justify less than minimum thickness

- ACI code clearly states that calculations need to be made if less than minimum thickness used
- Slabs with less than minimum thickness not recommended
- Estimate deflections even if minimum thickness exceeded (ACI 435R provides guidance)

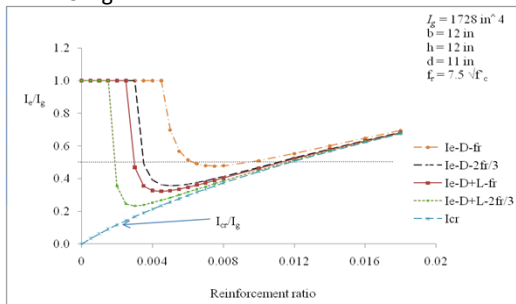
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Calculations made but ignoring cracking in slab

- Lightly loaded slabs sensitive to cracking
- Deflection underestimated if cracking ignored
- Slabs with low reinforcement ratio
-Substantial reduction in stiffness after cracking

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I_e/I_g vs Reinforcement Ratio



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Calculations made but ignoring effects of construction loading

- Dead load deflection should consider pre-loading due to construction loading
- I_e based on dead plus live load provides a reasonable estimate of the cracked stiffness

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Calculations made but ignoring effects of shrinkage restraint

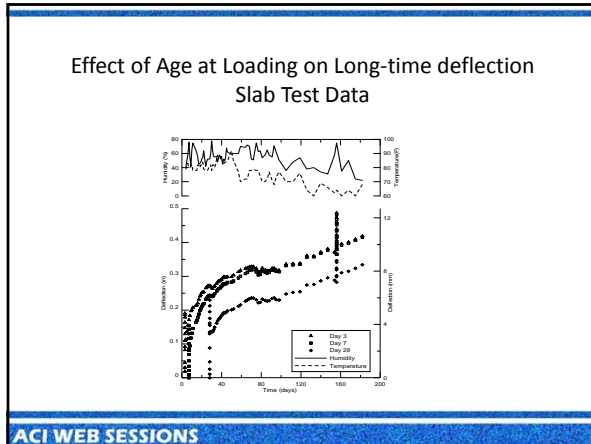
- Modulus of rupture – a material property based on flexural test of plain concrete specimen
- BUT cracking caused by both applied loads and shrinkage restraint (embedded reinforcement, stiff walls etc.)
- Account for this by using a reduced modulus of rupture ($f_r = 4\sqrt{f'_c}$ suggested, 1982)

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Calculations made but ignoring increase in creep due to early-age loading

- Long-time multiplier of 2 reasonable for slabs loaded at 28 days
- For loading at less than 7 days, an additional multiplier (1.25 to 1.50) should be used

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Applying minimum thickness to cases outside normal range

Examples:

- podium slabs (Bondy)
- excessive SDL
- very long spans

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		DL(psf)	LL(psf)	*LL red (psf)
R	Roof	20	20	20
4 th	PART	15	40	24
	FLOOR	25		
3 rd	PART	15	40	24
	FLOOR	25		
2 nd	PART	15	40	24
	FLOOR	25		
1 st	PART	15	40	40
	SLAB	125		
	MISC	10		
TOTAL		290	180	132

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Cantilevers

- Cantilevers susceptible to deflection problems- damage to exterior cladding
- Minimum thickness not covered in Table 9.5(c)
- Use Table 9.5(a) (one-way slabs) as guide
- Deflection due to bending of slab + rotation at support


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SUMMARY


- Punching Shear and Deflection Control
- Slab thickness (span-to-depth ratio) is the main issue
- Recommendations
 - Satisfy minimum thickness
 - Careful check of punching shear
 - Estimate deflections particularly if loading or span arrangements outside the "normal" range
 - careful evaluation and monitoring of construction

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