American Concrete Institute®
Advancing concrete knowledge

Analysis, Design, and Construction Practices in Environmental Engineering Concrete Structures, Part 2 of 2

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

ACI Web Sessions

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ACI Web Sessions

This ACI Web Session includes two 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.

Please enjoy the presentations.

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Analysis, Design, and Construction Practices in Environmental Engineering Concrete Structures, Part 2 of 2

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

Javeed Munshi has 20 years of engineering experience in design, evaluation and construction of concrete structures including heavy industrial (fossil and nuclear) power structures, bridges, underground structures (tunnels), buildings, and environmental concrete structures. He is currently a Senior Structural Engineer at Bechtel Power in Frederick, MD. Dr. Munshi is widely published and has conducted concrete design seminars and training for the American Concrete Institute, the Portland Cement Association, and the Concrete Reinforcing Steel Institute. He is a licensed professional engineer in states of New York and Wisconsin and a licensed structural engineer in Illinois.

Strength Requirements

Slides prepared by
Javeed Munshi, Ph.D., P.E.

Strength Requirements

- Chapter 8 – Analysis and Design Considerations
  - 8.1.1 - Design using Load factors and phi factors of Chapter 9
  - 8.1.2 - Appendix I (Allowable Stress Design Permitted only for nonprestressed members)
  - 8.3 - Approximate methods of analysis are permitted for nonprestressed members

Chapter 9 – Strength Requirements

- Chapter 9 – Strength

  Design Strength ≥ Required Strength

  ϕ (Nominal Strength) ≥ U

  Nominal Strength = Axial, shear or moment strength of section based on strength and sectional properties of concrete and strength, location and amount of reinforcement

Load Factors

Required Flexure Strength $\geq$ S_d *U*

$\Phi M_n \geq S_d (1.2M_D + 1.6M_L + 1.6M_F)$

Required Direct Hoop Strength $\geq S_d$ *U*

Excess shear carried by shear reinforcement

$\Phi V_s \geq S_d (V_u - \Phi V_c)$

Environmental Durability Factor -- S_d

$S_d = \phi f_y / \gamma f_s$

γ = Factored Load/Service Load

Flexural Sections:

f_s, max = $320/\beta\sqrt{(s^2 + 4(2+0.5d_b)^2}$……Normal Exp.

= $260/\beta\sqrt{(s^2 + 4(2+0.5d_b)^2}$……Severe Exp.

$\leq$ 36 ksi

β = (h - c)/(d - c) [use 1.2 for h≥16 in. and 1.35 for h < 16 in.]

Environmental Durability Factor -- S_d

$S_d = \phi f_y / \gamma f_s$

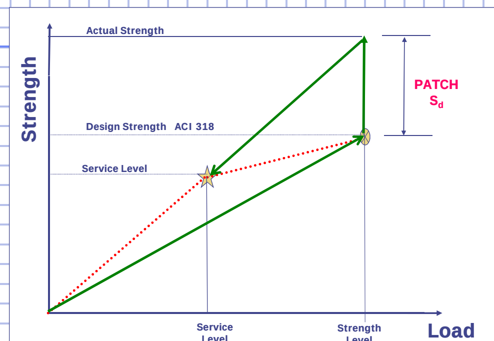
γ = Factored Load/Service Load

Direct/Hoop Tension: f_s, max = 20 ksi Normal

= 17 ksi Severe

Shear Reinforcement: **fs**, max = 24 ksi Normal

= 20 ksi Severe



Environmental Durability

- **Section 9.2.6 and 10.6.4**
  - **Keep cracks tight at serviceability to prevent leakage**
    - **Flexure**
    - **Direct Tension / Hoop Tension**
    - **Shear Reinforcement**

Env. Durability Factor S_d

[Kianoush & Atashi]

Bar Size	Exposure Condition	f_s	S_d
#3 - 5	Severe	22	1.75
	Normal	24	1.61
#6 - 8	Severe	18	2.14
	Normal	22	1.75
#9 - 11	Severe	17	2.26
	Normal	21	1.84

Design Loads

I. Floor Live Loads

A. Equipment and Stored Materials

B. Personnel and Transient Loads

Floor design loads should take into account:

1. Installation
2. Maintenance
3. Operation,

of equipment and materials.

Design Loads

II. Fluid Loads

A. Normal Fluid Levels

B. Overflow Fluid Levels

C. Internal Positive or Negative Air Pressures

1. Due to Actual Operations
2. Due to Rapid Filling or Rapid Draw down

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Design Loads

II. Fluid Loads

D. External Fluid Loads

1. Ground Water or Flood condition
2. Earth Loads
   a) Active, passive and at rest pressures
3. Buoyancy/Floatation

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Design Loads

III. Environmental Loads (as per local codes)

A. Wind

B. Snow

C. Thermal

D. Earthquake

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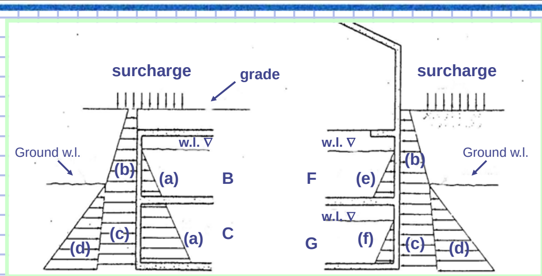

Pressures on walls – water treatment plant

(a) Inside water pressure
(b) Earth active or at rest pressure
(c) Submerged earth pressure
(d) Ground water pressure
(e) Filter box water & submerged filter material
(f) Clear well water pressure

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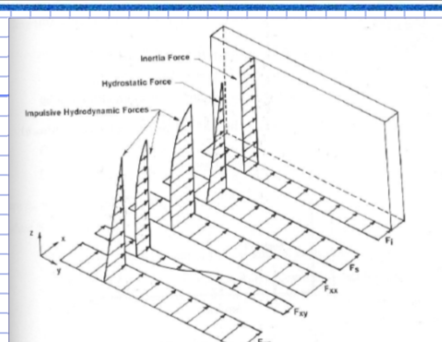

FIG. 4. Typical loads on tank walls.

Pressures on walls

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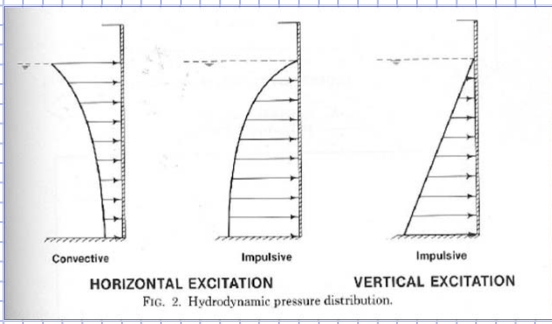

Hydrodynamic Pressure

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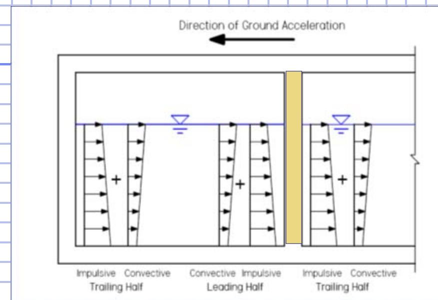

Hydrodynamic Pressure Middle (Common) Wall

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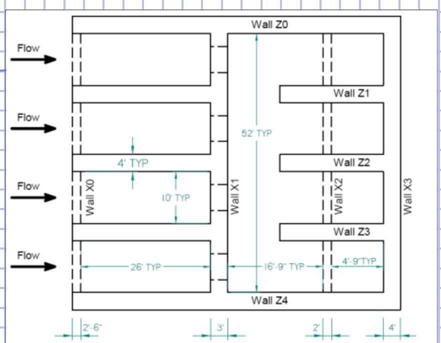

Hydrodynamic Pressure

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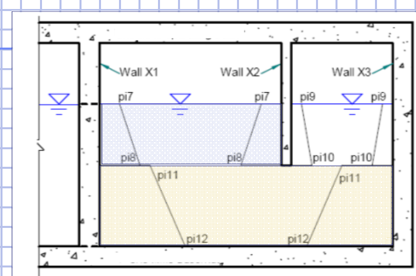

Hydrodynamic Pressure

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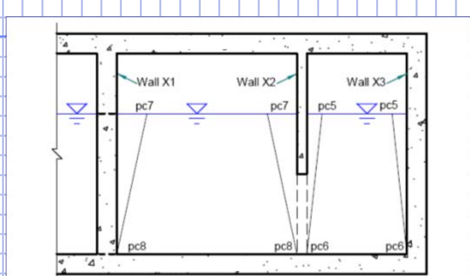

Hydrodynamic Pressure

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Movement Required for Active/Passive Pressure Development

Approximate Magnitudes of Movements Required to Reach Minimum Active and Maximum Passive Earth Pressure Conditions
From Clough and Duncan (1991)

Type of Backfill	Values of Y/H* Active	Values of Y/H* Passive
Dense sand	0.001	0.01
Medium-dense sand	0.002	0.02
Loose sand	0.004	0.04

*Y = movement of top of wall required to reach minimum active or maximum passive pressure, by tilting or lateral translation.
H = height of wall.

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Dynamic Earthpressure

- Active or Passive
- Soil – cohesive or cohesionless
- Water Table
- Rigid or Flexible Wall

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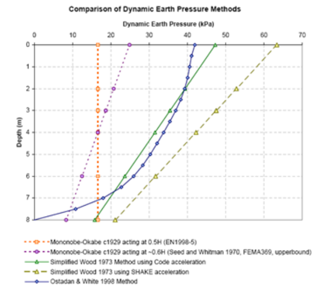

Figure 6. Comparison of calculated dynamic earth pressures for a rigid embedded wall, founded on rock. Sand with $\phi'=35°$, Dr =65%. PGA (rock) = 0.15g. PGA (soil) = 0.2g (UBC97), PGA (soil) =0.27g (SHAKE). NB: M-O interpretations for comparison only, not strictly applicable for this wall type.

Merrick Taylor, SECED Newsletter, Oct 09

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$$\Delta P_E = \gamma H^2 k_h F_p ,$$

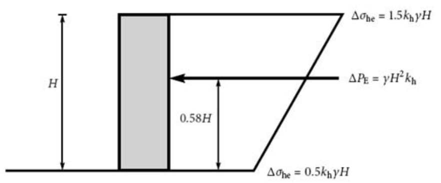

Figure 5. Dynamic earth pressure profile on rigid walls, after Matthewson et al. [26].

Fp = thrust factor = 1 for normal soils

Merrick Taylor, SECED Newsletter, Oct 09

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Saturated Soil Factor

Dry = 1.0

Dynamically pervious = 1.6

Dynamically impervious = 2.0 (pore pressure development)

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Flexible wall-backfill or wall-base

Dynamic earth pressure is considerably less

Fp could be 0.5 or less

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Possible Simplification

Seismic earth pressures can be neglected at accelerations below 0.3 g. This is consistent with the observations and analyses performed by Clough and Fragaszy (1977) and Fragaszy and Clough (1980), who concluded that conventionally designed cantilever walls with granular backfill could reasonably be expected to resist seismic loads at accelerations up to 0.5 g.

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Unified Design Approach

- Required strength unchanged
  - All load combinations of Sect. 9 apply
- Design strength
  - Design strength = ϕ (nominal strength)
  - Nominal strength does not change

Unified Design Approach

- To unify and simplify the overall design requirements
- To remove many of the inconsis-tencies

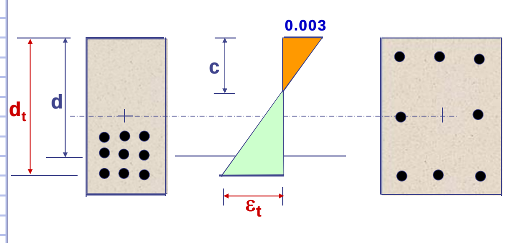
Unified Design Approach

- ϕ- factor determined by strain condition of a section at nominal strength, and not on the type of loading
- Strain condition defines the behavior

Unified Design Approach

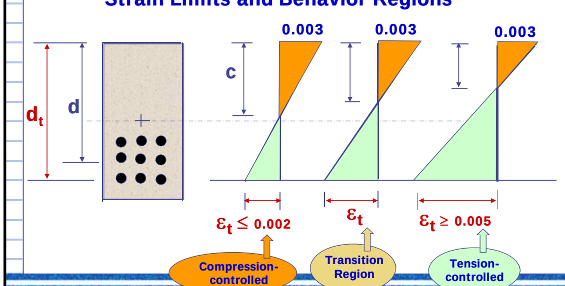
- **Terms used to define the strain condition**
  - ε_t – net tensile strain in extreme tension steel at depth d_t at nominal strength due to the factored loads, exclusive of effective prestress strain
  - d_t – distance from extreme compression fiber to extreme tension steel

Unified Design Approach



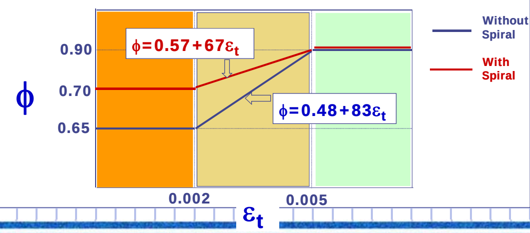
Unified Design Approach

Strain Limits and Behavior Regions



Unified Design Approach

Strength Reduction Factor ϕ



Unified Design Approach - Applicability

- Flexural and compression members
- Nonprestressed, prestressed and combinations
- Sections of any shape
- Composite sections
- Steel at various depths

Section 9.3 ϕ - Factors

- Shear and torsion = 0.75
- Bearing = 0.65
- Post-tensioned anchorage zones = 0.85
- Strut and Tie Models = 0.75
- Plain Concrete = NA

Section 9.2 Load Combinations

Eq. No.	Load Combination - ASCE 7	
9-1	1.4(D + L)	
9-2	1.2(D + F + T) + 1.6(L+H)+0.5(Lr or S or R)	
	1.2(D + F + T) + 1.6L+H +..	*see (d)*
	1.2(D + T) + 1.6(L+H)+..	see (e)
	1.2(D + F + T) + 1.6H +..	see *(e)*
	1.2(D + F + T) + H +..	see *(d)*
9-3	1.2D + L + 1.6 (Lr or S or R)	
	1.2D + 1.6 (Lr…)	*see (e)*

Section 9.2 Load Combinations

Eq. No.	Load Combination - ASCE 7	
9-4	1.2D ± 1.6W+ L+ 0.5(Lr or S or R)	
	1.2D ± 1.6W+ 0.5(Lr or S or R)	*see (e)*
9-5	1.2D + 1.2F ± E+1.6H+L+0.2S	
	1.2D + 1.2F ± E+H+L+0.2S	*see (d)*
	1.2D ± E+1.6H+L+0.2S	*see (e)*
	1.2D + 1.2F ± E+H+0.2S	*see (e)*
9-6	0.9D + 1.2F± 1.6W+1.6H	
	0.9D + 1.2F± 1.6W+H	*see (d)*

Section 9.2 Load Combinations

Eq. No.	Load Combination – ASCE 7	
9-6	0.9D + 1.2F± 1.6W	*see (e)*
	0.9D ± 1.6W+H	
	0.9D ± 1.6W+1.6H	
	0.9D ± 1.6W	*Not in ACI 350?*
9-7	0.9D + 1.2F± E+1.6H	
	0.9D + 1.2F± E+H	
	0.9D + 1.2F± E	
	0.9D ± E+H	
	0.9D ± E+1.6H	
	0.9D ± E	*Not in ACI 350?*

Example – Beam Design

M_u = 600 ft-kips (ϕ = 0.9)

f'_c = 4000 psi, f_y = 60,000 psi

Preliminary Design: Use As = 6 No 9 Bars and d = 30 in

Typ. Cover = 2.5 in.

$f_{s,\,max} = 320/\beta\sqrt{(s^2 + 4(2+0.5d_b)^2} > 36$ ksi,

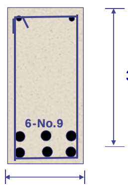
Use $f_{s,\,max}$ = 36 ksi.

$S_d = \phi f_y / \gamma f_s = 1.07$,

where γ = 1.4, β = 1.2 and

s = 3 in.

Use S_d = 1.1



Example – Beam Design

- a = 8.9 in.
- c = 10.4 in., dt = 31 in;
- ε_t > 0.005
- ϕ = 0.23+0.25/(c/dt) > 0.9
- ϕM_n = 0.9 x 6 x 60 (30-8.9/2)/12 =690 ft-kips
- $M_u = S_d$ x 600
  = 1.1 x 600 = 660 ft-kips < 690 ft-kips OK

Example – Slabs

Assuming γ = 1.4, β = 1.35 for h < 16 in.

s > 6 in. (typically)

$f_{s,}$ max = $320/\beta\sqrt{(s^2 + 4(2+0.5d_b)^2} < 30$ ksi,

$S_d = \phi f_y / \gamma f_s$ = 1.3 or Larger

Flexure and Axial Loads

Reinforcement Spacing For Appearance – New Eq. 10-7

$$s = 540/f_s - 2.5c_c$$

But not greater than 12 in

TABLE 7.12.2.1—MINIMUM SHRINKAGE AND TEMPERATURE REINFORCEMENT

Length between movement joints, ft	Minimum shrinkage and temperature reinforcement ratio: Grade 40	Grade 60
Less than 20	0.0030	0.0030
20 to less than 30	0.0040	0.0030
30 to less than 40	0.0050	0.0040
Greater than 40	0.0060*	0.0050*

*Maximum shrinkage and temperature reinforcement where movement joints are not provided.
Note: When using this table, the actual joint spacing shall be multiplied by 1.5 if no more than 50% of the reinforcement passes through the joint.

Concrete sections that are at least 24 in. may have the minimum shrinkage and temperature reinforcement based on a 12 in. concrete layer at each face. The reinforcement in the bottom of base slabs in contact with soil may be reduced to 50 percent of that required in Table 7.12.2.1.

William (Bill) Sherman is a principal structural technologist at CH2M HILL and a registered professional/structural engineer in a number of states. He has over 35 years of structural engineering experience, including over 20 years working on water/wastewater projects. He is the current chair of ACI Committee 350, and has previously participated on ACI Committee 301. Mr. Sherman's structural engineering experience includes planning, design, evaluation, and services during construction for numerous public works projects in the water resources and power industry. His experience includes developing design criteria, analyses, construction drawings, specifications, shop drawing reviews, quality assurance, and contract administration for newly constructed structures and modifications to existing structures.

Future Code Changes and Specifications

Slides prepared by
William C. Sherman , Chair ACI 350

ACI 350 Code

- Code is being actively updated
- Once balloted it goes first to ACI TAC review and second to 45-day Public Comment Period
- About 2 or 3 years before publication

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Adoption of ACI 318-08 revisions

- Since our code relies heavily on the ACI 318 code, we have updated the 350 code to incorporate ACI 318 changes that are appropriate for environmental engineering concrete structures
- ACI 350 will always lag ACI 318 code provisions since 318 must be finalized before 350 can adopt changes

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Major Revisions to ACI 350 Code

- Code Chapters have been re-organized
- Chapter 4 extensively revised to correspond with selection of Exposure Categories as has been done in ACI 318-08
- A new Chapter on Joints (CH 7)
- Some former Appendices have been incorporated into the main body of the code

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Chapter Reorganization

- New Chapter 7 on Joints
- "Details of Reinforcement" moved from CH 7 to CH 12 (with Development and Splices)
- Seismic Design moved from CH 21 to CH 13
- Appendix G for Circular Wrapped Prestressed Walls moved to CH 21
- Appendix H for Slabs on Soil moved to CH 22

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Chapter Reorganization (cont.)

- Appendix I for the Alternate Design Method moved back to Appendix A
- New Appendix B proposed to address the Strut & Tie method
- Other Chapters and Appendices are similar to previous but may have new numbers/ letters

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Chapter 4 - Durability

- Code requirements reformatted similar to ACI 318-08 to address “Exposure Categories and Classes”, such as freeze-thaw, sulfates, etc.
- Exposures modified some as required for environmental concrete structures
- Provisions for Supplementary Cementitious Materials are being expanded, with new upper and lower limits on SCMs unless the testing is conducted

Chapter 23 – Evaluation of Existing Structures

- ACI 318’s Chapter on existing structures emphasizes verification of strength
- ACI 350’s version now addresses “Strength Evaluation and Condition Assessment of Structures”

Significant proposals being debated

- New provisions for shrinkage and temperature reinforcement – to relate required reinf to degree of restraint in lieu of basing on joint spacing
- Proposal to remove restriction on using unbonded tendons for circumferential prestressed reinforcement
- Proposal to modify allowable stresses when using the Alternate Design provisions (ASD method) for better consistency with recent code provisions for strength

ACI 350.3, Seismic Design of Liquid-Containing Concrete Structures

- Being updated and issued simultaneously with the ACI 350 Code, to ensure compatibility of seismic requirements
- Seismic chapter in ACI 350 Code:
  - Special systems are similar to ACI 318 provisions
  - General provisions are more specific to applicability to tanks
  - Most special provisions for tanks are defined in Section 13.1

ACI 350.3 (cont.)

Some tweaking of seismic provisions:

- Importance Factors modified to be consistent with definitions in ASCE 7-05
- R-values modified but slightly higher than ASCE 7
- Vertical acceleration for rectangular tanks increased for consistency with circular tanks

ACI 350.1, Specification for Tightness Testing of Environmental Containment Structures

- Recently revised and has been approved for publication
- Major reformatting to convert from a “Standard” to a “Reference Specification”
- Some tweaking and clarification of provisions
- Provides test methods and default allowable percent loss values
- Does not define what structures must be tightness tested

ACI 350.X, Specifications for Environmental Concrete

- Based on ACI 301-05
- Provisions modified as for Environmental Concrete Structures, such as more restrictive durability requirements for concrete mixes
- Special provisions for prestressed tanks

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THANK YOU FOR YOUR TIME!

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Related Documents

Environmental Structures

- 350-06 Code Requirements for Environmental Engineering Concrete Structures
- 350.1-01/350.1R-01: Tightness Testing of Environmental Engineering Concrete Structures & Commentary
- 350.2R-04: Concrete Structures for Containment of Hazardous Materials
- 350.3-06 Seismic Design of Liquid-Containing Concrete Structures and Commentary
- 350.4R-04: Design Considerations for Environmental Engineering Concrete Structures

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