ACI 352-13 Guide to the Code for Evaluation, Repair, and Rehabilitation
ACI 352-13 Repair Code

This code provides:
• Evaluation procedures for existing concrete structures.
• Provides material and design requirements
• Guides the licensed design professional to bring existing concrete structures in compliance with building codes written for new construction.
Applicability

• Existing concrete structures,
• Concrete elements of buildings,
• Non building structures when required by the building official,
• Building foundation members,
• Soil-supported structural slabs,
• Concrete portions of composite members, and
• Precast concrete cladding that transmits lateral loads to diaphragms or bracing members.
Flowchart
Design Basic Code

Start of Project

- Verify that ACI 562 is applicable (§1.2)

Determine the original building code under which the structure was designed and constructed (§1.1.3)

Determine the current building code under which the new structures are designed and constructed (§1.1.3)

- Is there an existing building code adopted by the jurisdiction? (§1.1.2, §1.1.4)

  - Yes: Go to §4.1 and §4.2
    - Based on criteria in existing building code, select compliance method (§4.2.1)
      - Prescriptive Method
      - Work Area Method
      - Performance Method

  - No: Go to §1.3

- Perform evaluation as necessary in accordance with §4.3 and/or Ch. 6

- Is structure compliant with the original building code? (§1.3.3)

  - Yes: Is structure safe? (§1.3.4)
    - Yes: Use original building code and ACI 562 as design basis code
    - No: Use current building code and ACI 562 as design basis code
Responsibilities of the LDP

- Evaluation
- Specifying repair materials and details
- Developing quality assurance programs covering the repairs
- Advising the owner on future maintenance requirements
- Report identified unsafe structural conditions to the owner and jurisdictional authorities
4.2 Compliance method

• The 2012 IEBC describes three compliance methods:
  • Prescriptive method,
  • Work area method,
  • Performance method,

• The methods are summarized in Appendix C of this document. Chapter 34 of the 2012 IBC, which is applicable to existing buildings, includes a specified compliance method similar to the prescriptive method in the 2012 IEBC and an alternate compliance method similar to the performance method in the 2012 IEBC.
Preliminary evaluation

• The level and extent of the preliminary evaluation is subject to the professional judgment of the LDP.

• In some cases where the repair project will address structural damage, the extent of deterioration or deficiency and case of damage are clearly defined and known without significant evaluation.

• In other cases, a preliminary evaluation may suggest the need for a more detailed evaluation.

• The existing conditions, as well as any observed deficiencies or deterioration, should be considered in the course of a preliminary evaluation.
5.1.3 It is not permitted to use load factors and load combinations from the original design building code with strength reduction factors from this chapter. It is not permitted to use load factors and load combinations from this chapter with strength reduction factors from the original design building code.

5.1.6 When this code is the design basis code, loads shall be determined in accordance with ASCE/SEI 7, except that seismic loads shall be determined in accordance with ASCE/SEI 41.
5.2—Load factors and load combinations

• 5.2.1 Design of the repair shall account for existing loads on the structure; the effects of load removal; and the sequencing of load application, including construction and shoring loads, during the repair process.

• 5.2.2 Structural members and connections, whether being designed for a repair or being structurally evaluated, shall have design strengths at all sections at least equal to the required strengths calculated for factored loads and forces in such combinations as stipulated in this code.
If determined by the structural assessment that the strength of a structure is not in question, structural analysis is not required.
Fig. 6.1: Summary of evaluation methodology of ACI 562

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Preliminary Evaluation (§4.3)

1. Review of plans and available documents (§4.3.1).
2. Existing conditions visually or otherwise assessed for deterioration, damage, or distress (§4.3.3).
3. Preliminary analysis and evaluation to determine strength based on in-place or assumed geometry and material properties (§4.3.4).

Is a structural evaluation necessary based on the requirements of §6.1.2 or §6.1.3?

Yes

Structural Evaluation (Ch. 6)

1. Structural assessment of affected members (§6.2).
2. Obtain material properties as required to perform evaluation (§6.3 or §6.4).
   Perform the following as required based on the results of the assessment.
3. Structural analysis of existing member or structure (§6.5).
4. Serviceability evaluation of member or structure (§6.6).
5. Strength evaluation by load testing if appropriate (§6.8).

Based on results of evaluation proceed with design of repair or rehabilitation if required. Perform analysis for design in accordance with §6.7.

No
Requirements for Structural Evaluation

- Effects of material degradation,
  - loss of concrete strength from chemical attack;
  - freezing and thawing;
  - loss of steel area due to corrosion or other causes.
- Effect of deterioration on the ductility of the section

The strength or serviceability of a structure may be compromised by spalling, excessive cracking, large deflections, or other forms of degradation.
A structural assessment shall document existing conditions as necessary to perform a structural analysis.

If an analysis is required, the structural assessment shall document:

a) As-measured structural member section properties and dimensions.

b) The presence and effect of any alterations to the structural system.

c) Loads, occupancy, or usage different from the original design.

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Existing Condition

• Physical Condition
• Load Paths
• As-built Data
• Structural Members – Existing Conditions
• Material Properties
• Additional Considerations
• Seismic Resistance Assessment
Material properties

• Material properties shall be obtained from available drawings, specifications, and other documents for existing construction.

• If such documents do not provide sufficient information to characterize the material properties, this information shall be obtained from the historical data provided in Tables 6.3.1a through 6.3.1c, or

• Determination of material properties in accordance with the requirements of 6.3.5, or both.
6.3.5 When properties are to be determined by in-place testing, the locations and numbers of material samples shall be defined by the licensed design professional. The number of samples shall not be less than required by the test standard.

• The focus of the prescribed material testing should be on the principal structural members and specific properties needed for analysis.

• The licensed design professional should determine the appropriate number and type of testing needed to evaluate the existing conditions.

• Core drilling should minimize damage of the existing reinforcement and should generally occur at locations where the coring will least affect the member strength.
Structural analysis of existing structures

- The licensed design professional is responsible for determining the appropriate method of analysis
  - Linear Elastic
  - Non Linear
  - Other

- External effects
  - Pre stressing
  - Material volume changes
  - Temperature variations
  - Differential foundation movement
Analysis

- Must consider the load path
- Consider three dimensional distribution of loads and forces in the complete structural system
- Consider the effects of previous repairs and of any previous structural modifications on the behavior of the structure.
Structural serviceability

• Serviceability analysis must be performed based on in place geometry and properties

• Issues:
  • Deflections
  • Vibrations
  • Leakage
  • Objectionable cracking

• Performance criteria defined by professional
  • Floor deflection criteria in ASCE/SEI 7
  • Vibration criteria given in Murray et al. (1999).
Seismic analysis of repaired structure

- Consider interaction of structural members and nonstructural components that affect the response of the structure to seismic motions

- Existing, repaired, and added supplementary members assumed not to be a part of the seismic-force resisting system shall be permitted provided their effect on the response of the system is considered and accommodated in the repair design. Consequences of failure of structural members that are not a part of the seismic-force-resisting system and nonstructural components shall be considered.

- The method of analysis shall consider the structural configuration and material properties after repair.
Strength evaluation by load testing

• Load testing is permitted
  • Can verify strength of a structural system
  • To achieve a reliable estimate of short term strength
  • May provide most effective means of verifying strength

• Use when:
  • Inconclusive field assessments
  • Unknown effects of existing conditions

• For example, as defined in ACI 437-13, can be performed to determine that the service load deflection and cracking are acceptable.
The strength and serviceability behavior of existing concrete structures can be evaluated using information from the original design and construction.

When deterioration is present, the strength and stiffness may no longer be adequately predicted using the assumptions from the original design.

Deterioration and changes to member stiffness associated with repair work may result in the redistribution of internal forces to other portions of the member or to other members not anticipated by the original design.

The deterioration may also cause inelastic behavior, with permanent deformations or cracking. As a result, load redistribution may increase forces and stresses in portions of members less affected by deterioration.
…Strength and serviceability

- The LDP develop repair design and construction procedures that consider the stiffness, loading, internal forces, and deformations of both the existing and repaired structure.

- The repair design must consider if the distribution of internal forces and deformations in the members and structure are significantly affected by the repair process and accommodate or alter the repair process accordingly.

- The effect of the repair process on the stiffness of the structure and member under repair also needs to be considered.
Strength and serviceability

• Designed For:
  • Adequate Stiffness to limit
    • Vibrations
    • Cracking
    • Deformations

• Considerations given to:
  • In place internal forces
  • Some forces and deformations may be locked in by repair

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• Bond > 1.5 (Bond Force)
• The measured bond strength shall not be less than the lower of the required bond strength or the tensile strength of the existing concrete.
• Testing: Follow ASTM C 1583
• Supplementary reinforcement permitted
Consider compatibility of repair materials:
- Dimensional
- Bond
- Durability
- Mechanical
- Permeability
- Electrochemical

Consider global performance of structure
Issues to consider

- Corrosion
- Mechanical Damage
- Delaminated Concrete
- Strength

Proper development needed:

- ACI 318-11
- ACI 440.1R
- ACI 440.2R

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Supplemental post-tensioning can introduce additional moment, shear, and axial forces within the existing structure that should be considered in the design and detailing of the repair.

The internal forces induced by the supplemental post-tensioning can be significant. For statically indeterminate structures, restraint to post-tensioning deformations can result in significant internal forces.
Repair using fiber-reinforced polymer (FRP) composites

- FRP is permitted in conformance to ACI 440.6
- Used externally or internally

Attention to:
- Strength increase limits
- Service limits
- Design Properties
ACI 562, Section 8.1.2, requires the repair program address:

- **Compatibility** of the repair materials
- **Interaction of repair materials** with the existing structure
- **Durability** of the repair materials and the existing structure
- Anticipated **maintenance**
• QA/QC is required as in any construction project!!
• Inspected as per existing building code
  • No building code? - Design professional shall propose inspection scheme (Section 10.1C)
  • Include inspection procedures in contract documents
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Reparaciones Estructurales:
SOLUCIONES INNOVADORAS

Project Examples
352 Guide Example Projects

- Example #1 - Typical Parking Garage Repair
- Example #2 - Typical Façade Repair
- Example #3 - Adaptive Reuse of Historic Depot
- Example #4 - Parking/Plaza Slab Strengthening
- Example #5 - Precast/Prestressed Double-Tee Repair
Example #1
Typical Parking Garage Repair
Parking Example

60% Delaminated Area

20% Delaminated Area
Description

- The structure is a **two-story, enclosed parking garage** located in the northern U.S.
- The garage, constructed in the 1960s, measures approximately 240 x 150 ft (73.2 x 45.7 m) in plan.
- The lower level is on ground, and the **middle level and roof consist of reinforced concrete flat slabs** with drop panels.
- The middle-level deck was covered with an asphalt topping.
- **No design information or drawings** were available.
At the middle-level deck, the owner noted potholes and unevenness in the asphalt topping and water leakage through cracks.

A few small pieces of concrete had fallen from the underside of the slab.

The project was initiated to determine the current condition of the garage and to develop a plan and urgency for garage maintenance.
Based on discussions with the building officials, the building codes adopted by the jurisdiction were determined.

- **Jurisdiction** - northern U. S. city.
- **Existing Building Code** - Covered in Chapter 34 of 2006 IBC.

ACI 562 supplements the existing building code and governs in all matters pertaining to concrete members in existing buildings, except wherever ACI 562 is in conflict with the requirements in the existing building code, in which case Chapter 34 of the 2006 IBC governs.

- **Design Basis Codes** - Based on a preliminary evaluation as described in, the design basis code was determined to be the 1961 UBC.
Existing Conditions

Existing structural geometry.
- The existing structural geometry, including typical dimensions and member sizes, was measured on-site.

Existing concrete condition.
- On the top surface of the middle-level deck, the asphalt topping was removed, exposing the top concrete surface. Exposed concrete surfaces were visually surveyed for types and patterns of distress and deterioration (ACI 201.1R, “Guide for Conducting a Visual Inspection of Concrete in Service”).
- Concrete surfaces were selectively sounded by chain drag, hammer tapping, or tapping with a reinforcing bar to estimate the extent of delaminated concrete.
• The underside of the middle-level slab was 10 to 20% delaminated or spalled due to corrosion of the embedded reinforcement, with greater damage observed in Slab Area 1.

• Just above the slab, the bases of columns had small areas of delamination due to corrosion of the embedded reinforcement mostly in Slab Area 1.

• The upper portion of the middle-level slab was heavily contaminated with chloride, greatly exceeding the corrosion threshold value (ACI 222R), particularly in areas that were delaminated, spalled, or cracked.
The reinforcing layout and condition was documented at a few typical locations by measurement of exposed bars, magnetic survey, and exploratory chipping to expose bars.

Bar size, spacing, and concrete cover were determined for top and bottom reinforcement in column strips and middle strips at exterior columns, first interior columns, and interior columns.

Surface corrosion and some section loss were documented for the top slab bars in Slab Area 1 (ACI 364.1R).
The middle-level slab and the columns were in compliance with the 1961 UBC without consideration of current deterioration.

As there was substantial documented top surface concrete deterioration in Slab Area 1 but significant effort would be necessary to accurately determine the structural effects of this deterioration, the top slab reinforcing bars were conservatively judged to be debonded at delaminations and therefore structurally ineffective.

Accordingly, the shear and flexural strength in Slab Area 1 was found to be substantially less than that required by the 1961 UBC.

Slab Area 1 was determined to be structurally deficient and unsafe.
Effect of unsounded concrete in reinforcement

**Diagram Explanation:**

- **LOCAL DELAMINATION**
  - Reduced or no bond between reinforcement bar and concrete.
  - Slab representation.

- **LARGE DELAMINATION**
  - Reduced or no bond between reinforcement bar and concrete.
  - Slab representation.

- **REINFORCING BAR**

- **UNSOUND CONCRETE**

- **$l_d$** - Tension development length per ACI 318-05.

- **$l_d$** - Is the embedded length at least $l_d$?
As there is no effective top reinforcement near the columns, \( d \), the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement, becomes very small (essentially the distance between the bottom of the slab and the bottom reinforcement), substantially reducing the slab shear capacity and resulting in an unsafe condition, as determined relative to 1961 UBC. 

Moment design:

\[
M_{\text{typ int}} = \frac{1}{12}wl^2 < M_{\text{all}}
\]

\[
M^+_{\text{typ int}} = \frac{1}{12}wl^2 < M^+_{\text{all}}
\]

Current moment capacity considering deterioration -

\( M_{\text{all}} = 0 \) due to debonding of top reinforcement, which means that \( M_{\text{typ int}} = 0 \).

which causes \( M^+_{\text{typ int}} \) to increase to \( \frac{1}{8}wl^2 > M^+_{\text{all}} \), which is an unsafe condition based on UBC 1961.
• The **owner was notified of the safety concerns.**

• **Shoring was promptly installed** to support Slab Area 1 to address
  the safety concern and to allow continued access to the garage
  until repairs could be installed.

• Loose concrete was promptly removed from the underside of the
  slabs.

• **Substantial structural damage** per ACI 562 commentary is defined
  by the 2012 IEBC, which states that *substantial structural damage*
  *refers only to vertical elements in the gravity load-carrying system*
  *or lateral force-resisting system*; that is, the columns in this structure.
  As the columns had only small localized concrete deterioration, it
  was determined that there is no substantial structural damage
  based on the 2012 IEBC.
Based on the simplifying preliminary assumption made by the LDP that the top slab reinforcement in Slab Area 1 is totally debonded and ineffective, the slab was deemed unsafe.

As no excessive cracking or deflections were noted, the slab is apparently still performing satisfactorily in spite of the extensive deterioration and therefore the preliminary assumption is conservative particularly for areas with little to minimal deterioration.

Structural elements outside of Slab Area 1 have some concrete deterioration but were not considered by the LDP or the authorities having jurisdiction as unsound or structurally deficient.

Code changes in detailing and other requirements make it difficult if not impossible to bring existing concrete structures into full compliance with current code requirements.
Concrete strength.
• Concrete core samples were extracted and tested in compression to determine the slab concrete compressive strength. The strength values were consistent with the strength assumed in the preliminary analysis.

Reinforcing steel layout and strength.
• Reinforcing steel spacing and cover were determined with ground-penetrating radar and confirmed at exposed bars and exploratory openings. Exposed reinforcing bars were examined for identification marks that might indicate the steel yield strength. No marks were found.
• Coupons were removed from reinforcing bars and tested in tension to determine the steel yield strength. The strength values were consistent with the strength assumed in the preliminary analysis.
• The repaired middle-level slab was analyzed as a two-dimensional frame based on the in-place material properties.

• The analysis considered the structural repair process, including the effects of the sequence of load application and material removal.

• It was also assumed that the replacement concrete would be fully bonded to the existing concrete and, hence, that there would be full composite action between repair materials and existing materials.
Redistribution

- It was assumed that approximately 50 percent of the negative moment capacity had been lost, and the increased steel and concrete stresses in the positive moment region were calculated.

- It was then assumed that the shoring supported the slab during construction, such that no loads from construction were resisted by the slab.

- When the construction had been completed and the shoring removed, it was assumed that the topping weight and the design live load were supported by the repaired composite section.

- The capacity of the repaired section was examined and was determined to have adequate strength to resist the design loads.
• Newer codes, such as the ACI 318-05 referenced by the current building code (2006 IBC), specified that a portion of the unbalanced slab moments must be transferred into the column by eccentricity of the shear, thus increasing the maximum punching shear.

• A close visual inspection of the top and bottom surfaces of the middle-level slab around the first interior columns, where the unbalanced slab moments are greatest, did not detect any cracking that might be indicative of distress due to inadequate punching shear capacity.
Slab repairs were designed according to the provisions of the 1961 UBC. **Two repair options** for deteriorated concrete on the top surface were discussed with the owner:

1. Removal and replacement of deteriorated concrete only on the top slab surface.
2. Removal and replacement of the top 3 to 4 in. (75 to 100 mm) of concrete in the entire area.
• Chloride-contaminated concrete around the top reinforcing mat is removed and replaced with uncontaminated concrete with low permeability, improving durability and reducing future maintenance.

• The new concrete will have similar or slightly enhanced properties compared to the existing concrete.

• After concrete removal work has been completed, the exposed concrete surfaces will be cleaned and a suitable bonding procedure will be used to attain the minimum required bond strength and ensure composite behavior under service loads.

• Existing reinforcing bars, except for those embedded in columns, are removed and replaced with new epoxy-coated reinforcing bars, replacing bars with reduced cross-sectional area. Because the new bars are uncontaminated and coated with epoxy, their resistance to corrosion is much improved, improving durability and reducing future maintenance of the slab system.

• Existing bars to remain are cleaned and coated with a corrosion-inhibiting material.
• Top reinforcing bars with shallow cover can be relocated downward in the slab for increased corrosion protection cover, assuming that the slab still has adequate calculated shear capacity with the decreased the effective depth and that additional bars are added as necessary to provide adequate calculated flexural capacity.

• New reinforcing bars are fully encapsulated and developed in the replacement concrete.

• The repaired slab will have similar or larger strength and stiffness to the originally constructed sections.

• Due to the new uncontaminated concrete with low permeability, and the epoxy-coated reinforcement, new surface coatings such as a traffic-bearing elastomeric coating or a surface sealer were not recommended, reducing initial and maintenance costs.
Option 2 Disadvantages

- The perimeter of the partial-depth replacement area must be located and detailed to account for shear and moment transfer and reinforcing steel development.
- The slab will need to be shored prior to the slab removal and remain shored until the new slab concrete has been placed and cured.
- Cracks that may form in the replacement concrete should be sealed.
- This repair option has a higher initial cost as compared to Option 1.
• Only deteriorated concrete is removed and replaced, limiting repairs and repair costs to current requirements.
• Re-entrant comers will be avoided in both the repair and existing concrete.
• After concrete removal work has been completed, the exposed concrete surfaces will be cleaned and a suitable bonding procedure will be used to attain the minimum required bond strength and ensure composite behavior under service loads.
• Existing reinforcing bars that are exposed in removal areas will be cleaned and coated with a corrosion-inhibiting material to reduce ongoing corrosion in and around the replacement concrete areas.
• New epoxy-coated reinforcing bars will be lapped with existing bars that are exposed in removal areas and have lost structurally significant cross-sectional area.
• Discrete galvanic anodes will be installed around the perimeter of slab concrete replacements to reduce corrosion in the existing concrete around the concrete replacements. To function properly, the anodes must be attached to uncoated portions of the reinforcing bars in the removal areas before the bars are coated with a corrosion-inhibiting material.
...Disadvantages

- **Except at repair locations, chloride-contaminated concrete will remain in place**, resulting in some ongoing corrosion activity and concrete and steel deterioration requiring periodic maintenance repairs. The corrosion reduction measures incorporated into the repair program should significantly reduce ongoing corrosion activity and periodic repair requirements.

- The LDP must establish limits for concrete removal and monitor the removal work so that shoring can be installed before the limits are exceeded.

- The LDP must monitor the concrete removal work for loss of reinforcing steel development and possible short-term and long-term structural implications, and for possible structurally significant loss of reinforcement cross-sectional area, as determined by the LDP.

- The LDP must determine if unsafe conditions may exist and if temporary shoring should be installed.
The repair specifications included quality assurance and control measures for material approvals and field verification of quality. The specified quality control measures and construction observations were performed during the construction, including the following:

- Review of **material submittals** and reinforcement shop drawings for Slab Area 1.
- **Visual inspection** of the work in progress.
- **Sounding of concrete surfaces** to remain to determine if all loose concrete was removed prior to repair.
- Observation of the prepared concrete surfaces and of the concrete placement and curing operations.
- **Testing of repair concrete**, including slump, temperature, and compressive strength.
- Bond strength testing of in-place repair concrete to confirm that the bond strength was at least 1.5 times greater than the calculated design bond strength.
Periodic maintenance

- Periodic maintenance requirements were discussed with the owner during the selection of the most appropriate repair concepts. A schedule of recommended monitoring and possible maintenance requirements was provided to the owner at the conclusion of the repair construction, including the following:
  - Periodic inspections every 3 to 5 years to monitor the condition of the garage.
  - Limited concrete deck repairs every 5 years.
  - Limited repair of the traffic-bearing elastomeric coating every 3 to 5 years to address areas of high wear such as near the garage entrance/exit.
  - Top coating the traffic-bearing elastomeric coating and restriping the garage every 15 to 20 years.
Example #4
Parking/Plaza Slab Strengthening
Description of Structure

• The facility consists of an **11-story office building** over a **two-story, 100,000 ft² (9300 m²) parking structure** supporting an open-air plaza. The building is located in the northern U.S., and construction was completed in 2013.

• The supported **garage slab** is a **reinforced concrete flat plate**, and the **plaza slab** is a **reinforced concrete flat slab with drop panels**.

Project Initiation and Objectives

• Shortly after construction was completed and the certificate of occupancy was obtained, **excessive deflections and top surface cracking** were noted on the supported garage slab.

• The project was initiated to determine the causes of the cracking and deflections and the overall safety of the as-built structure.
LEVEL LL-1 SLAB DOES NOT EXTEND OVER THIS AREA, WHICH IS FILLED WITH SOIL.

CONCRETE BEAM BELOW SLAB

CONCRETE COLUMN, TYP

CONCRETE WALL

RAMP DOWN

CONCRETE WALL
Governing Building Code

• **Jurisdiction** - Northern U. S. city.

Existing Building Code - The jurisdiction had not adopted the International Existing Building Code (IEBC), thus repairs to existing structures are covered in Chapter 34 of 2012 IBC. The 2012 IBC states that “Work performed in accordance with the International Existing Building Code shall be deemed to comply with the provisions of this chapter.” (3401.6 Alternative Compliance). Accordingly, the 2009 IEBC is permitted to serve as the existing building code.
Based on a preliminary evaluation as described in the Preliminary Evaluation section of this example and considering the requirements of the existing building code, the design basis code was determined to be the 2009 IBC and, by reference, ACI 318-08.
• **Document review** - The design drawings, construction documents, and various reports were available and reviewed by the LDP. The construction documents and reports confirmed that the concrete and reinforcing bars met the specified material properties.

• **Existing site conditions** - Existing structural geometry. The existing structural geometry, including typical dimensions and member sizes, was measured on-site. The thickness of the drop panels in the plaza slab was measured to be approximately 6 in. (150 mm) compared to the design thickness of 14 in. (355 mm).

• **Existing concrete condition** - The top surface cracks on the supported garage slab were mapped and the top surface elevations were surveyed to document the slab deflections.

• **Reinforcement** - The reinforcement layout was documented using ground penetrating radar (GPR) supplemented by exploratory chipping at isolated locations to expose reinforcing bars and confirm the GPR findings. The investigation revealed areas having over 4 in. (100 mm) of concrete cover to the top reinforcing bars compared to the as-designed cover of 3/4 in. (19 mm).
A preliminary analysis was performed for a few locations of the supported garage slab and the plaza slab to estimate the decrease in the as-built strength compared to the design strength due to the documented construction deficiencies:

- Decreased drop panel thicknesses and
- Increased concrete cover over the top reinforcing bars.

The 2009 IBC was assumed to be the design basis code for the purposes of the preliminary evaluation.
Effects of shallow drop panels

• The shallow drop panel condition on the plaza slab decreased the effective depth of the slab reinforcement $d$, the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement, from the design value of 26 3/4 in. (680 mm) to the as-constructed value of 18 3/4 in. (475 mm), a decrease of approximately 30%.

• The slab flexural capacity is directly and indirectly related to $d$. The measured decrease in $d$ resulted in a calculated flexural deficiency of 30 to 40%.
The slab punching shear capacity is also directly related to \( d \). The measured decrease in \( d \) resulted in a **calculated shear deficiency of 30\%**.

\[
\begin{align*}
\varphi V_n &= \varphi (4) \left( \sqrt{f'_c} \right) b_o d \\
\varphi V_n &= \varphi (0.33) \left( \sqrt{f'_c} \right) b_o d
\end{align*}
\]

where \( \varphi \) is the strength reduction factor for shear.

\( V_n \) is the nominal punching shear strength of the section.

\( b_o \) is the perimeter of the critical section for shear in slabs.

Refer to Fig. 15.4 for an illustration of \( b_o \) and \( d \).

ACI 318-08, Eq. 11-35 (in.-lb units)

ACI 318-08, Eq. 11-35 (SI units)
Structural sections – Drop Panel

AS-DESIGNED

3/4" (19 mm) CLEAR COVER

COLUMN STEEL NOT SHOWN FOR CLARITY

24" TO 30" (610 to 760 mm)

AS-BUILT

3/4" (19 mm) CLEAR COVER

COLUMN STEEL NOT SHOWN FOR CLARITY

24" TO 30" (610 to 760 mm)
Reinforcement Cover

AS-DESIGNED

24" TO 30"
(610 TO 760 mm)

3/4" (19 mm)
CLEAR COVER

3/4" (19 mm)
CLEAR COVER

10" (250 mm)

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AS-BUILT

24" TO 30"
(610 TO 760 mm)

2" TO 4" (50 TO 100 mm)
CLEAR COVER
(3" [75 mm] COVER DEPICTED HERE)

3/4" (19 mm)
CLEAR COVER

10" (250 mm)
Effects of excessive cover

• The excessive cover for the top reinforcing bars in the supported garage slab similarly decreased the slab d from the as-designed dimension of 8 3/4 in. (220 mm) to as little as 5 3/4 in. (145 mm), a decrease of approximately 35%.

• Flexural and punching shear capacities were adversely affected, with a calculated flexural deficiency of 15 to 45% and a calculated shear deficiency of 18 to 46%.
Safety Concerns

• The LDP determined that the cracking and deflections of the supported garage slab and the gross deviations of the construction from the design were significant safety concerns.

• The LDP determined that both slabs had sufficient calculated capacity to support the estimated dead loads, but did not have sufficient calculated capacity to support the estimated dead loads and the design live loads, even if the strength reduction factors for evaluation permitted by ACI 562 Section 5.4 were used.

• The LDP advised the owner that the supported garage slab and the plaza should be immediately removed from service or shored to grade.
Effects of steel bracket or reinforced concrete column capital

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• **Compliance method.** For this project, the LDP elected to use the prescriptive compliance method in the 2012 IEBC.

• **Design basis code.** The construction deficiencies do not constitute substantial structural damage, as defined by the 2012 IEBC. Therefore, the design basis code for the required strengthening repairs was the 2009 IBC. (404.4, 2012 IEBC)

• **Structural deficiency.** A structural evaluation was necessary due to the structural deficiencies determined in the preliminary analysis.
Structural assessment

- Existing structural geometry. The existing structural geometry was documented in more detail than was done for the preliminary evaluation (4.2.1).
- All column spacings, column dimensions, and drop panel dimensions were measured.
- The slab thickness was determined with GPR. The reliability of the measurements was confirmed by physical measurements at several holes drilled through the slab.
Concrete strength.
- The concrete compressive strength was assumed to be the design strength, as confirmed by the construction testing laboratory reports that were available.

Reinforcing steel layout and strength.
- Reinforcing steel spacing and cover were determined at all locations with GPR and confirmed at exploratory openings. Bar sizes were also measured at the exploratory openings.
- The reinforcing steel tensile strength was assumed to be the design strength, as confirmed by the mill certificates in the construction records.
Strengthening concept 1

• **At locations with excessive concrete cover.** On the supported garage slab, the top portion of the slab concrete would be removed and reconstructed with new top bars correctly placed and vertical shear connectors crossing the bond line between the existing and new concrete.

• After the removal work, but before the reconstruction work, the slab would be unloaded by jacking so that the entire slab design load, including dead load and live load, would be transferred to the new repaired slab after the repairs have been installed, cured, and the jacks removed.

• This approach would restore the supported garage slab to its as-designed configuration, thus restoring its design capacity.
Strengthening concept 1

• **At locations with thin drop panels.** For the plaza slab, steel brackets would be installed on the columns at the underside of the slab.
  
  • The steel brackets would move the critical punching shear section further from the column, increasing the perimeter of the section, $b_o$, and thus the punching shear capacity.

  • The brackets also would move the critical negative moment section further from the column, decreasing the design negative moment to the moment capacity of the slab section with the reduced drop panel thickness.
...Strengthening concept 1
Strengthening concept 2

• Reinforced **concrete column capitals** would be **constructed** on the columns at the underside of both supported slabs. At a few short span conditions where there was no positive moment, of the supported garage slab. **Supplemental drop panels** would be **constructed on the underside**.

• At a few locations on the **underside of the plaza slab**, **carbon-fiber-reinforced polymer (CFRP) strips** would be installed as supplemental flexural reinforcement to allow some moment redistribution from the negative moment regions at the columns.
• **Punching shear capacity.** The column capitals move the critical punching shear section further from the columns, increasing the perimeter of the section and the punching shear capacity. The supplemental drop panels increase the effective slab depth, further increasing the punching shear capacity.

• **Design negative moment** The column capitals also move the critical negative moment section further from the columns, decreasing the design negative moment (Fig. 15.6). The supplemental drop panels increase the effective slab depth, increasing the negative moment capacity (Fig. 15.8).

• **Moment redistribution.** The CFRP reinforcement on the underside of the slab increases the positive moment capacity of the slab and allows some redistribution of negative moments to the positive moment regions.
Strengthening concept 2

\[ R_{SLAB} = (1/4) w l_1 l_2 \] - Slab reaction into column

\[ M_{w,A:B} = M_k - R_{SLAB} l_{A:B} + (1/2) w l_{A:B} l_2 = M_k - (1/2) w l_{A:B} l_2 (l_1/2 - l_{A:B}) \]

\[ M_{w,REP} = M_k - R_{SLAB} l_{REP} + (1/2) w l_{REP} l_2 = M_k - (1/2) w l_{REP} l_2 (l_1/2 - l_{REP}) \]

where
- \( l_1 \) is the length of the slab span in the direction of the moments being determined, measured center-to-center of supports
- \( l_2 \) is the length of the slab span perpendicular to \( l_1 \), measured center-to-center of supports
Evaluation of strengthening concepts

The preliminary details of the strengthening concepts were determined based on the findings of structural analyses performed in the Structural Evaluation section, and cost estimates were obtained for both strengthening concepts. 

**Strengthening Concept 2 was estimated to cost approximately 20% of the estimated cost of Strengthening Concept 1.**

Other factors such as construction scheduling and sequencing were not substantially different for the two concepts on this project. Using cost as the key differentiator, the owner elected to pursue Strengthening Concept 2.
SUPPLEMENTAL CONCRETE COLUMN CAPITAL - PLAN
Note: Verify locations of column reinforcing before fabricating capital reinforcing assembly.

EXISTING COLUMN
EPOXY GROUT DOWELS INTO COLUMN, 6" (150 mm) MAX. ON CENTER, 5 1/4" (13 mm) MIN. EMBEDMENT
No. 5 (No. 16) BAR ASSEMBLY
NEW COLUMN CAPITAL
18" OR 24"
(460 mm OR 610 mm)
3/4" (19 mm) COVER
2 1/2" (13 mm)
5 1/4" (130 mm)
MB1

ROUGHEN COLUMN SURFACE TO MINIMUM APITUDE OF 1/4" (6 mm) OR CSP 10. SANDBLAST CLEAN

EXISTING CONCRETE DECK ASSEMBLY CONNECTION

PROVIDE FULL CONTACT BETWEEN SLAB SOFFIT AND TOP OF NEW COLUMN CAPITAL BY PLACING CONCRETE THROUGH HOLES IN SLAB OR DRYPACKING TOP OF CAPITAL

1/2" (13 mm (5 mm))

TOP BARS CONTINUOUS EACH FACE, WELDED TO COMPANION BARS AT CORNERS.

4 - No. 4 (4 - No. 13) HOOP BARS AT 6" (130 mm) ON CENTER

3 - No. 4 (3 - No. 13) STRAIGHT BARS EACH FACE OF CAPITAL

SUPPLEMENTAL CONCRETE COLUMN CAPITAL - SECTION

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The effects of the repair process and the installed repairs were analyzed using the two dimensional plate models of the as-built slabs. The analysis considered the current condition of the slabs, including the cracking, the deflected shape, and the stresses in the slab due to dead loads. The stress increases due to live load in the repaired existing slabs were also evaluated. The adequacy of the bond strength at the interface between the repair material and the existing concrete was evaluated based on 17.5.4 of ACI 318-11.

The analysis assumed full composite action at the repair material interface.
The strengthening requirements of the repairs were determined based on the findings of the analyses of the original design, the as-built construction, and the repair design. The stiffening effects of the repairs were also evaluated. The repair design included the following details and considerations.

- The column capitals were designed and detailed to integrate and act compositely with the columns and the slab above.
- The column surfaces were intentionally roughened and hoop reinforcement was included in the capitals to transfer forces into the columns by shear friction. Self-consolidating concrete (SCC) was used to facilitate filling of the forms to the slab soffit.
- The supplemental drop panels were also designed and detailed to integrate and act compositely with the columns and slab above.
- Reinforcing steel extended between the column capitals and the drop panels, and the capitals and drop panels were placed monolithically. Epoxy-grouted dowels were installed in the slab soffit to transfer the horizontal flexural shear by shear friction. SCC was used to facilitate filling of the forms to the slab soffit.
• The CFRP strips were designed as a supplemental strengthening measure, based on the recommendations of ACI 440.2R.

• The structural strength of the repaired slab was adequate to carry the factored loads specified in the current building code without the CFRP strips.

• The CFRP strips were used to increase the calculated slab positive moment capacity to account for the increased positive moments due to moment redistribution that occurred before the repairs were installed, and to limit further deflections from these moments.

• The CFRP strips were detailed and bonded to the slab soffit per the manufacturers recommendations and ACI 440.2R. Bond pull off testing was performed to verify the bond.
• The injection of epoxy into cracks on the top surface of the supported garage slab was requested by the owner and also served to increase the slab stiffness and seal the cracks against intrusion of water and deicing salts into the concrete, improving the durability of the slab.

• A **traffic-bearing elastomeric coating was applied on the top slab surface in the negative moment regions** around the columns on the supported garage slab to prevent the intrusion of water and deicing salts into cracks that may not have been sealed and to minimize the intrusion of water and deicing salts into the concrete and improve the durability of the slab.

• As the column capital and drop panel repairs and the CFRP reinforcement were all installed on the undersides of the slabs, the repairs are not directly exposed to the harsh top slab surface service environment and the repairs were judged to be durable.
...Construction

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...Construction
CFRP installation
Epoxy Injection of Cracks

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The LDP prepared contract documents that specified repair materials that satisfied governing regulatory requirements and conveyed necessary information to perform the work.

The contract documents included the **minimum requirements for shoring and bracing for all phases** of the repair project,

It included requirements for the contractor to submit shoring documents that were **signed and sealed by an LDP**.
The contract documents required the contractor to monitor the construction for any conditions that were not consistent with the available information or that might affect the short-term or long-term safety of the structure, including the possible need for additional temporary shoring or bracing.

Requirements for environmental issues, such as allowing water with debris to flow into floor drains or off of the site and disposal of construction debris, were specified in conformance with local ordinances.
Quality Assurance

- The repair specifications included quality assurance and control measures for material approvals and field verification of quality. The specified quality control measures and construction observations were performed during the construction, including the following:
  - Review of material submittals and reinforcement shop drawings.
  - Visual inspection of the work in progress at critical stages of the repair.
  - Observation of the prepared concrete surfaces and comparison with ICRI concrete surface profiles (ICRI No. 310.1R-08) to verify that minimum roughness had been achieved.
  - Observation of the installed reinforcement.
  - Periodic inspection and pullout testing of epoxy-grouted dowels in the slab soffit in accordance with ACI 355.4.
• Observation of the concrete placement and curing operations.
• Testing of repair concrete, including slump flow, air content, temperature, and compressive strength.
• Impact-echo testing to verify the continuity between the slab soffit and the new capitals.
• Observation of the surface preparation and installation of the CFRP strips.
• Impact-echo testing was performed on the top slab surface over drop panel and capital repairs to detect possible gaps between the slab soffit and the repairs.
• Some areas with gaps were detected.
• Open joints between the slab soffit and drop panel and capital repairs were injected with epoxy to fill the joints and bond the repairs to the soffit. Continuity was confirmed by impact-echo testing.
Impact Echo testing

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Epoxy Leakage

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After the repairs had been installed, a representative portion of the repaired supported garage slab was evaluated by load testing to demonstrate the strength of the repaired slab.

The test area was selected based on typical repairs in the area and ease of setting up and running the test.
Test procedure

• ACI 562 references ACI 437-13 for load testing, which is meant to read as ACI 437.2-13. The 2009 IBC, the design basis code, references ACI 318-08, which includes Chapter 20, Strength Evaluation of Existing Structures.

• Based on ACI 562 Section 1.1.7, ACI 562 governs for all matters pertaining to evaluation and shall govern when in conflict with other referenced standards.

• Accordingly, the monotonic load test procedure described in ACI 437.2-13 was used for the evaluation.

• The monotonic load test was selected after consultation with the Contractor’s available means, methods, and familiarity with the monotonic test.
Monotonic load test

- Monotonic load test protocol has been used for several decades for the structural evaluation of concrete structures. The procedure basically involves loading the structure in a monotonic manner by gradually applying the load until reaching the test load magnitude, which is maintained for 24 hours.

- Measurements are recorded before any load is applied, after each load increment, when the maximum load is achieved, after 24 hours of sustained loading, and 24 hours subsequent to the removal of the test load.

- The structure is evaluated based on the maximum recorded deflection and the amount of deflection recovery. Monotonic loading can be achieved using dead weights or hydraulic jacks.
Arrangement of jacks
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Load Test