Column strip positive moment region:

It was determined conservatively that 15 percent of the bottom slab area was deteriorated and accordingly reinforcement in that area was debonded. If deterioration did not exist, the slab capacity in that region was calculated to be: ϕR_n = 133 ft-kip (180 kN·m). The slab capacity must be reduced by 15 percent to 113 ft-kip (153 kN·m) and additional moment demand from the negative region of the top slab must be added to determine the current condition.

$$M_{u,cs}^{+} = 123 \text{ ft-kip} + (285 \text{ ft-kip} - 275 \text{ ft-kip}) = 133 \text{ ft-kip}$$

$$(M_{u,cs}^{+} = 167 \text{ kN·m} + (386 \text{ kN·m} - 373 \text{ kN·m}) = 180 \text{ kN·m})$$

This will result in a demand-capacity ratio:

$$U_c/\phi R_{cn} = 133 \text{ ft-kip}/113 \text{ ft-kip} = 1.18 < 1.5$$

$$(U_c/\phi R_{cn} = 180 \text{ kN·m}/153 \text{ kN·m} = 1.18 < 1.5)$$

A.3.2

Per ACI 562 Section A.3.2, the section is safe and further assessment per Sections A.4 through A.9 is required.

Demand (U_c)	Design capacity	Actual capacity	Demand/capacity $U_c/\phi R_{cn}$
$M_{u,cs}^{+}$ = 133 ft-kip (180 kN·m)	ϕR_n = 133 ft-kip (180 kN·m)	ϕR_{cn} = 113 ft-kip (153 kN·m)	133 ft-kip/113 ft-kip = 1.18 < 1.5 (180 kN·m /153 kN·m = 1.18 < 1.5)

Per the commentary for the definition of damage, deterioration from aging should not be considered as damage. The columns had only small localized concrete deterioration and the check of Section A.5.2 of ACI 562 was not done in this example.

A.5.3

For Area 2, the slab is considered safe per ACI 562, Section A.3.2, and accordingly Section A.5.3 or A.5.2 may be checked to determine if strengthening is required. The following table lists the demand and capacity at critical sections in a typical interior two-way slab of the parking structure using Eq. (RA.5.3).

Location	Reinforcement	Demand, U_s $A_{s,req'd}$ in.^{2} (mm^2)	Capacity, R_a $A_{s,prov.}$ in.^{2} (mm^2)	U_s / R_a
Column strip	A_s^-	12.5 (8065)	13.5 (8710)	0.93 < 1
	A_s^+	6.8 (4387)	7.2 (4645)	0.94 < 1
Middle strip	A_s^-	5 (3226)	6 (3871)	0.83 < 1
	A_s^+	5 (3226)	6 (3871)	0.83 < 1

The slab two-way shear capacity was calculated at 89 psi (0.61 MPa), which is less than the allowable required by code of $0.03f_c'$= 90 psi (0.62 MPa) and therefore, increasing slab shear punching strength was not required.

A.5.3

ACI 562 Section A.5.3 commentary states that, "using the allowable design is inconsistent with the reliability principles of current strength design provisions. To adequately address safety, consideration should be given to verification using A.5.2 and a check of seismic resistance using ASCE/SEI 41."

Seismic resistance is not an issue in the region and was excluded from the analysis.

ACI 562 Sections A.5.1, A.5.2, or A.5.3 per Table A.2.3 were used to verify the structural adequacy in Area 2. This was not required to be performed at Area 1 because it was already determined to be unsafe and would be repaired to be in compliance with the current code.

Report to owner

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 parking structure until repairs could be installed. Also, loose concrete was promptly removed from the underside of the slabs. As immediate safety concerns were promptly addressed, it was not necessary to notify the authorities having jurisdiction.

The LDP considered the following factors:

1.5.3

a. Based on the simplifying preliminary assumption made by the LDP that the top slab reinforcement in Slab Area 1 is over 60 percent debonded and ineffective, the slab was deemed unsafe.
b. As no excessive cracking or deflections were noted, the slab is apparently still performing satisfactorily despite the extensive deterioration and, therefore, the preliminary assumption is conservative particularly for areas with little to minimal deterioration.
c. Structural elements outside of Slab Area 1 have some concrete deterioration but were not considered by the LDP or authorities having jurisdiction as unsound or structurally deficient.

A.2.4, A.2.5, RA.5.1

d. 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. Although Area 1 has unsafe structural conditions, the structure has demonstrated historical structural reliability having been in service for more than 50 years, is located in a region where seismic activity is minimal, and is to be repaired; therefore, full compliance with detailing requirements was not necessary.

The LDP determined, and the authorities having jurisdiction agreed, that the design basis code should be 1961 UBC, with the provision that where possible, the slab in Slab Area 1 should be brought into conformance with the requirements of the current building code 2015 IBC.

STRUCTURAL ASSESSMENT

Existing conditions

6.1.1, 6.2.2, 6.2,3, 6.2.4

Existing structural geometry—The existing structural geometry was documented in more detail than was done for the preliminary assessment (4.1.1).

a. All column spacing, column dimensions, and drop panel dimensions were measured.
b. The slab thickness was determined with ground-penetrating radar and confirmed by physical measurements at holes drilled through the slab.

6.4.2.1, 6.4.3.1

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.

6.2.4, 6.4.4.1

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. Additional areas were investigated for reinforcing layout, sizes and, cover using magnetic survey, cover meter, and exploratory chipping to expose bars. The extent of measurement of expanded from the preliminary measurements and the results were consistent with the preliminary findings. Therefore, the same assumptions for the preliminary assessments were used for the final assessments.

6.4.5.1

Coupons were removed from reinforcing bars and tested in tension to determine the steel yield strength. Strength values were consistent with the strength assumed in the preliminary analysis.

Structural analysis for repair design

1.3.8.2, 5.2.3

Loads factors and load combinations—The loads, load factors, and load combinations are as specified in ASCE/SEI 7-10.

6.5.1, 6.5.2, 6.5.3, 6.5.5, 6.5.7, 6.7.1, 6.7.2, 6.7.3

Analysis—In the preliminary assessment, the direct design method was used to determine the elevated slab moments, whether the slab was safe, and if strengthening was required. For the final assessment, a three-dimensional finite element model was developed to confirm the preliminary assessment findings, to evaluate the proposed repair procedure, and to assess the repaired system potential performance.

For the current state, the LDP used the actual reduced slab thicknesses due to delamination and the actual material properties obtained from in-place testing removing eight cored cylinders. The LDP took caution not to drill through existing reinforcement by locating the bars before drilling. Additional nondestructive testing was performed using rebound hammer to collect additional data. Correlation was established between the nondestructive data and test results obtained from the extracted cylinders.

The finite element analysis confirmed the preliminary assessment calculation for Area 1 and Area 2. The calculated demand of the gravity loads based on the current design code to the obtained capacity from the software output exceeded 1.5 for Area 1 indicating an unsafe condition. For Area 2, the demand-capacity ratio calculated per A.5.1 did not exceed 1.0; therefore, the structure did not require strengthening.

The elevated slab was then reanalyzed considering the structural repair process, including the effects of the sequence of shoring for Area 1, load application, and material removal for both areas. The final finite element analysis assumed that the replacement concrete would be fully bonded to the existing concrete and, hence, there would be full composite action between repair materials and existing materials. The demand-capacity ratio of both repaired areas (Areas, 1 and 2), obtained from the final analysis, was below 1.0; therefore, the repair design deemed theoretically satisfactory. Note that the asphalt overlay was removed and replaced with concrete overlay. The difference in the material unit weight was considered in the final finite element analysis.

7.4.1, 7.4.1.1

Bonding of the new concrete to the existing concrete was critical to satisfactory performance of the repaired structure. The horizontal shear demand at the interface of the repair and existing concrete was calculated based on the loads and combinations described previously. ACI 562 Section 7.4.1 requires the bond strength demand to be at least equal to the bond strength capacity ($\nu_u \geq \phi\nu_{ni}$).

Shear was calculated at face of columns and at face of drop panels, change in slab thickness. Based on the applied load, the ultimate shear stress at face of columns and at face of drop panels were calculated at 22 and 28 psi (0.15 and 0.19 MPa), respectively.

7.4.1.2

These stress values were compared to the shear stress values in Table 7.4.1.2 of ACI 562. Because the calculated ultimate stress values were smaller than 30 psi (0.21 MPa), interface reinforcement was not required and bond-integrity testing as specified in the construction documents must be performed.

As part of the field assessment, pulloff testing of the substrate determined that the substrate had adequate strength to achieve the required bond strength. Based on required bond strength, the LDP concluded that the required bond could be attained by chemical or mechanical means with proper surface preparation and repair material application (ICRI 310.1R discusses surface preparation for repair of deteriorated concrete). Therefore, no supplemental reinforcement was required. Refer to Example 7.2 of this guide for more detailed information on the testing requirements, and pulloff test to ensure adequacy of the bond between overlay and substrate.

Area 1

6.5.4

While the extent of debonding of the top reinforcing steel was unclear, the analysis also considered the possible redistribution of load effects and resulting increases in concrete and steel stresses due to the deterioration and subsequent installation of shoring, as follows:

a. It was assumed that approximately 60 percent of the negative moment capacity had been lost, and the increased steel and concrete stresses in the positive moment region were calculated.
b. It was then assumed that the shoring supported the slab during construction, such that no loads from construction were resisted by the slab.
c. When construction had been completed and the shoring removed, it was assumed that the topping weight and design live load were supported by the repaired composite section. The capacity of the repaired section was examined and determined to have adequate strength to resist design loads.

Area 2

6.5.4

The extent of debonding area of the top reinforcing steel was substantially smaller than what was observed in Area 1; therefore, redistribution of load effects was not considered and subsequently installation of shoring was not considered:

a. It was assumed that approximately 20 percent of the negative moment capacity had been lost, and the increased steel and concrete stresses in the positive moment region were calculated and found to be negligible.
b. When the construction was completed, 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 determined to have adequate strength to resist the design loads.

A.3.2, A.5.1

Consideration of punching shear—The LDP understood the 1961 UBC considered only vertical shear, or punching shear, transfer from the slab to columns. Newer codes, such as ACI 318-14 referenced by the current building code (2015 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.2.4, A2.5, RA.5.1

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. Although ACI 318-14 predicts an inadequate punching shear capacity at some columns, the LDP determined that because the slab prior to deterioration had performed satisfactorily for 50 years, and that it was being repaired back to original strength, that it was unnecessary to bring the punching shear capacity into conformance with provisions of the current building code.

1.5.3.a, 1.5.3.b, 1.5.3e, 1.5.3g

The LDP provided the owner with a basis of design report providing a description of the structure, identifying the structural system, and listing the codes used for the design and construction of the structure. The basis of design report also included documentation of unsafe structural conditions in the work area as presented previously and identified members that required strengthening. Strengthening options were presented to the owner along with the advantages and disadvantages of each option along with the recommendation of the LDP.

DESIGN OF STRUCTURAL REPAIRS AND DURABILITY

Slab area 1

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.

Option 2 was recommended for the reasons described in the following.

Option 2 advantages—

8.4.3, 8.4.4

a. Chloride-contaminated concrete around the top reinforcing mat is to be removed and replaced with uncontaminated concrete with low permeability, improving durability and reducing future maintenance.

7.5.2

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

7.3.2, 7.3.3, 7.4

c. 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. Surface roughness of the exposed concrete surfaces will be specified per a Concrete Surface Profile number from ICRI Guideline No. 310.2R or some other means.

7.3.2, 8.4.2, 8.4.4

d. Existing reinforcing bars, except for those embedded in columns, are to be 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 to be cleaned and coated with a corrosion-inhibiting material.

8.2.1, 8.2.2

e. Top reinforcing bars with shallow cover can be relocated downward in the slab for increased corrosion protection cover. This is assuming that the slab still has adequate calculated shear capacity with a decreased effective depth, and that additional bars are added as necessary to provide adequate calculated flexural capacity.

7.6.3.3

f. New reinforcing bars are to be fully encapsulated and developed in the replacement concrete.

7.1.1, 7.2.1, 7.2.2

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

8.5.1

h. 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.

i. The new slab will have similar or enhanced fire resistance rating compared to that of the existing slab.

7.9.1, 7.9.2, 7.9.3

j. The higher initial cost of this repair option will be at least partially offset by lower future maintenance costs.

k. As this repair area is at the parking structure entrance, less future maintenance also equates to fewer, shorter parking structure closures and less user inconvenience.

Option 2 disadvantages—

7.3.2, 7.3.3, 7.6.3.3

a. The perimeter of the partial-depth replacement area must be located and detailed to account for shear and moment transfer and reinforcing steel development.

9.2.5, 9.2.6

b. 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.

c. Cracks that may form in the replacement concrete should be sealed.

8.3.1

d. This repair option has a higher initial cost as compared to Option 1.

Slab Area 2 and columns

Replacement of deteriorated concrete only was recommended in this slab area, as the partial-depth replacement option recommended for Slab Area 1 was not a cost-effective approach for the limited concrete deterioration in this area. Similarly, the columns have very limited concrete deterioration and only replacement of deteriorated concrete was recommended.

This limited approach has the following advantages and disadvantages.

Advantages—

a. Only deteriorated concrete is to be removed and replaced, limiting repairs and repair costs to current requirements.

7.6.6

b. Reentrant corners will be avoided in both the repair and existing concrete.

7.3.2, 7.4

c. After concrete removal work has been completed, the exposed concrete surfaces will be cleaned and a suitable bonding procedure used to attain the minimum required bond strength and ensure composite behavior under service loads. Surface roughness of the exposed concrete surfaces will be specified per a Concrete Surface Profile number from ICRI Guideline No. 310.2R or some other means.

8.4.2, 8.4.4

d. 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.

7.6.3.1, 8.4.2, 8.4.4

e. New epoxy-coated reinforcing bars will be lapped with existing bars that are exposed in removal areas and that have lost structurally significant cross-sectional area.

8.4.1, 8.4.4

f. 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.

8.2.2

g. As much of the existing concrete will remain, the as-built reinforcing steel cover generally will not be modified.

7.5.2

h. The replacement concrete will have similar or slightly enhanced properties compared to the existing concrete.

7.6.3.3

i. The existing and new reinforcing bars will be developed in the existing concrete, the repair concrete, or both.

7.1.1, 7.2.1, 7.2.2

j. The repaired sections will have similar strength and stiffness to the originally constructed sections.

8.3.1, 8.3.2

k. Existing cracks will be addressed prior to the installation of the traffic-bearing elastomeric coating. New cracks that may form in the replacement concrete may be sealed by the traffic-bearing elastomeric coating or will be addressed by future maintenance repairs.

8.5.1, 8.5.2

l. A traffic-bearing elastomeric coating will be applied on the repaired slab surface to drastically reduce moisture penetration into the slab concrete and reduce ongoing corrosion activity in the remaining existing concrete and concrete replacements. The membrane will extend several inches up the column bases so that moisture on the deck surface cannot directly access the column concrete.

7.9.1, 7.9.2, 7.9.3

m. The repaired slab will have similar or enhanced fire resistance rating compared to that of the existing slab. This repair approach has a relatively low initial cost but periodic maintenance repairs will be necessary. It is a very cost-effective approach to address the present condition of the parking structure.

7.2.2

n. Analysis of the middle-level slab determined that the slab concrete remaining after the assumed extent of removal of deteriorated concrete can safely support the dead and construction live loads during the repair installation and its portion of the long-term dead and live loads after the repairs have been completed.

***Disadvantages*—**

8.4.3

a. Except at repair locations, chloride-contaminated concrete will remain in place, resulting in some ongoing corrosion activity with 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.

9.2.2, 9.2.5

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

9.2.2, 9.2.5, 9.2.6, 10.2.3

c. 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.

Slab soffit repairs

1.5.1, 1.6.1

The replacement of deteriorated concrete only was recommended on the slab soffit throughout the supported slab areas.

Construction specifications

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

The LDP used ACI 563 as a source for construction specifications. The specification sections that were referenced included:

Section 1—General requirements

Section 2—Shoring and bracing

Section 3—Concrete removal and preparation for repair
Section 5—Reinforcement and reinforcement support
Section 6—Conventional concrete mixtures
Section 7—Handling and placing of conventional concrete
Section 9—Crack repair by epoxy injection

The repair work did not require any formwork, therefore, Section 4—Formwork, was not referenced. Based on the size of the repairs, a conventional concrete was specified by the LDP in place of a proprietary material or shotcrete.

ACI 563, "Specifications for Repair of Concrete Buildings," addresses conditions that are unique to the project. The standard has mandatory and nonmandatory requirements checklists at the end of the standard to help the specifier submit as complete a specification as possible. For the parking structure slab repair, only a few sections from the mandatory checklists are extracted to include in the Project Contract Document:

a. Section 1.5.1.1—State the maximum dead and live loads and any temporary reduction in loads, to be permitted during repair and after completion of repair program, in concert with the requirements of 2.1.1.1.
b. Section 1.5.4.1—Show the demarcation line of the project location, specific work areas, and adjacent construction.
c. Section 1.8.2.1—Identify work to be performed by certified personnel.
d. Section 3.1.1.2—Provide the surface profile and remove laitance, debris, and bond-inhibiting materials.
e. Section 3.1.3.1—Indicate testing locations, type, number, and frequency of tests.
f. Section 3.2.1.1—Select the means and methods for concrete removal that will minimize damage to the structure and bruised surfaces on the concrete substrate that remains within and adjacent to the work areas.
g. Section 3.2.1.5—State the required surface profile.
h. Section 3.3.1.1—Show the required depth of concrete removal.
i. Section 3.3.4.2—Indicate that tensile pull-off tests shall be performed at specified locations in accordance with ASTM C1583/C1583M.
j. Section 5.2.1.2(b)—Indicate ASTM specification to which epoxy-coated reinforcing bars are to conform.
k. Section 6.2.2.6(d)—State the chloride exposure classification for are of work.
l. Section 6.2.2.7—Indicate the specified concrete compressive strength f_c' for the work.
m. Section 7.1.2.2—List the information in 7.1.2.2(a) to 7.1.2.2(g) that is to be submitted.
n. Section 9—Repair of cracks by epoxy injection in accordance with ACI 503.7.

Construction

9.2

The LDP monitored the construction for unexpected conditions that may affect the short-term or long-term safety of the structure. Temporary shoring or bracing may be necessary (Sections 6.2.1 and 6.2.4).

9.4.1

Environmental issues, such as allowing water with debris to flow into floor drains or off of the site and disposal of construction debris, will be specified in conformance with local ordinances.

Quality assurance

1.5.1, 10.2.1, 10.2.2, 10.4.1

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:

a. Review of material submittals and reinforcement shop drawings for Slab Area 1.
b. Visual inspection of the work in progress.
c. Sounding of concrete surfaces to remain to determine if all loose concrete was removed prior to repair.

10.2.3

d. Observation of the prepared concrete surfaces and of the concrete placement and curing operations.

10.3.1

e. Testing of repair concrete, including slump, temperature, and compressive strength.

f. Bond strength testing of in-place repair concrete to confirm that the bond strength was in accordance with Table 7.4.1.2 of ACI 562.

PROJECT CLOSE-OUT

Periodic maintenance

R1.5.3k, 8.1.2

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:

a. Periodic inspections every 3 to 5 years to monitor the condition of the parking structure.

b. Limited concrete deck repairs every 5 years.

c. Limited repair of the traffic-bearing elastomeric coating every 3 to 5 years to address areas of high wear such as near the parking structure entrance/exit.

d. Top coating the traffic-bearing elastomeric coating and restriping the parking structure every 15 to 20 years.

Record documents

1.6.3, 1.5.3d, R1.5.3j

The owner was provided with copies of the project and construction documents and the recommended monitoring and maintenance program.

Chapter 13: Project Example 2—Typical Façade Repair

Description of structure

The structure is a 28-story residential tower located in the northern United States. The building, constructed in the 1970s, measures approximately 80 x 90 ft (24.4 x 27.4 m) in plan, as shown in Fig. 13.1. The north and south elevations are cast-in-place reinforced concrete shear walls with 1 in. (25 mm) deep reveal strips at every floor line. The east and west elevations consist of exposed slab and column edges with glass-and-metal curtainwall infills. Several tiers on the east and west elevations have reinforced concrete balconies that cantilever out from the building. The original design drawings were available.

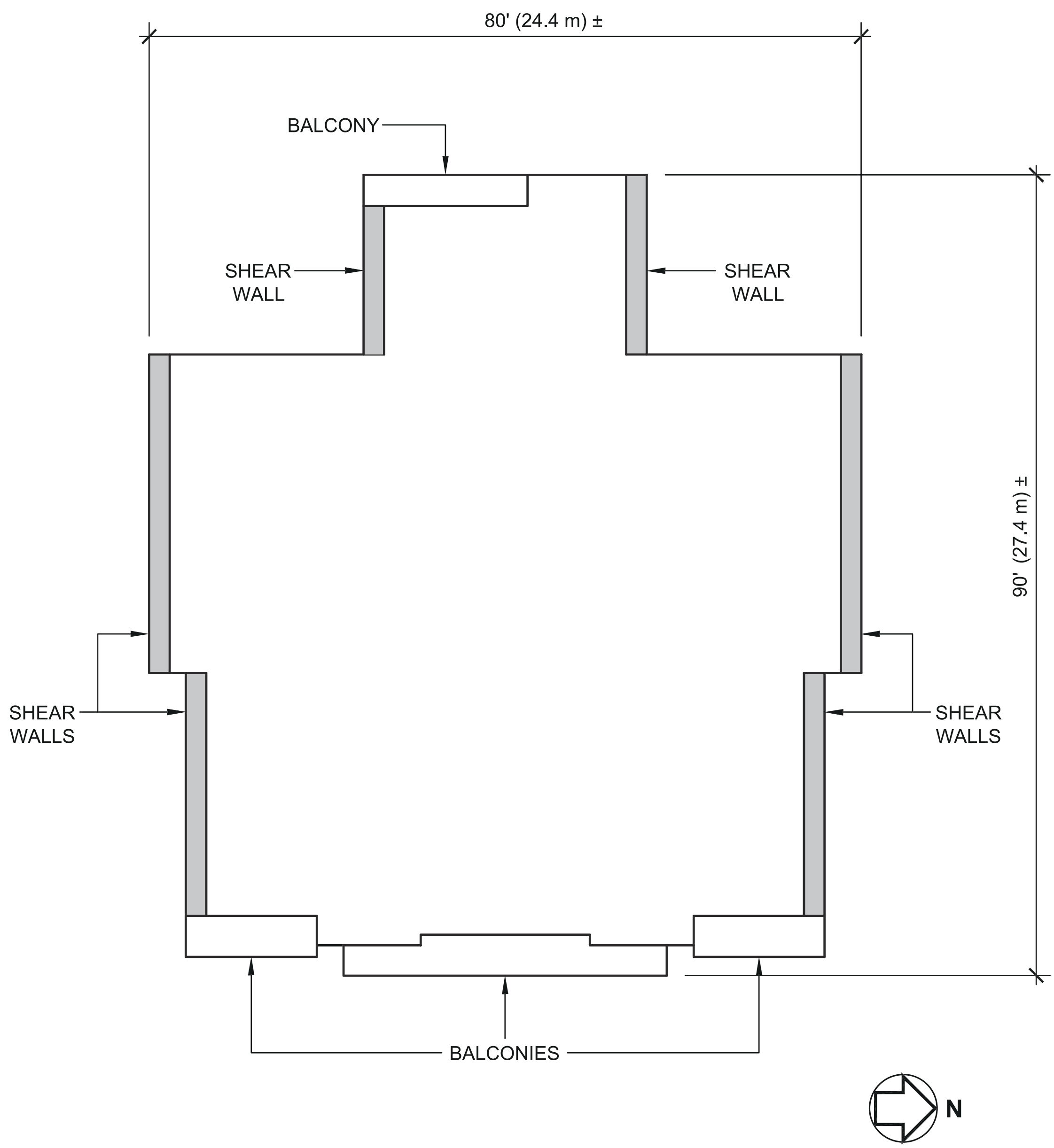

Fig. 13.1—Plan of residential tower.

ACI 562-19 provision numbers applying to each section of text are shown in red at the top right of each paragraph.