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EXCESS CAPACITY IN HISTORIC AMERICAN REINFORCED CONCRETE FLOORS

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Abstract. *The introduction of reinforced concrete as a structural material in American practice circa 1900 created unique challenges in design and analysis. Engineers and builders had to simultaneously determine proper constituent materials for the concrete, the relation between materials proportions and strength, and formulas for sharing load between steel and concrete. Early on there were only local building codes, which often treated concrete as masonry. This was followed, starting in 1909, by the reports of the "Joint Committee" which served as a de facto national code for concrete; the American Concrete Institute code was first independently published in 1941 and has been updated regularly since.*

All of the early codes, specifically including the Joint Committee report and the ACI code, used allowable-stress design with a linear-elastic model of flexure. Load-and-resistance factor design using an ultimate-strength model was first introduced in the US as an alternate method in the 1956 ACI code, was elevated to equal status with the 1963 code, and was made the standard in the 1971 code with the linear-elastic allowable-stress model as an alternate. Since the ultimate-strength model is generally accepted to more accurately represent flexure in reinforced concrete, the allowable-stress alternate was little used after the transition.

Many details have changed minimally since the 1910s, including provisions for T beams that take advantage of the slab for additional compression flange area. The calculation of shear capacity in both concrete and web reinforcing changed minimally except for the transition from allowable-stress to load-and-resistance-factor design.

Re-analysis of extant concrete structures designed before the 1956 code generally shows a pattern of significant excess capacity in flexure and limited excess capacity in shear. For members that are generally not controlled by one-way (beam) shear (two-way slabs and one-way slabs) the increase in flexure capacity governs; for members where one-way shear is a controlling factor, the small increase, no change, or decrease in shear capacity governs.

Much of the current literature on evaluating the capacity of extant concrete structures focusses on strength degradation from material changes such as rusting rebar and from overall material changes such as carbonation. The presence of excess capacity should also be considered, and it is possible based on review of the governing codes and design methods to determine general patterns as to how much excess capacity may be expected.

1 INTRODUCTION

Accurate of analysis of existing buildings depends in part on understanding their original design. In the case of older buildings with reinforced-concrete structure, there are significant differences between current design and analysis methods and those in use prior to the 1970s. Concrete as a structural material developed later than steel, almost entirely after 1900, which means that processes to create formal standards were in place from the beginning.

The details of material use, construction economics, and design standards differ from one country to another, so this study represent US practice. (The data and formulas used are given in their original units, with metric conversions following in parentheses.) However, it is likely that a similar analysis can be made in any country where reinforced-concrete use began in earnest before 1930.

There is no national building code in the US, as all such laws are enacted at the state level; the *International Building Code* (IBC), starting in 2000, was the first base code generally adopted by states across the entire country. As a result, there were differing standards for structure and other technical issues until national organizations created specific material codes. For example, structural steel was standardized with the 1923 release of the first edition of the *Standard Specification for Structural Steel for Building*s by the American Institute of Steel Construction.

From 1909 to 1941, the most prominent national standard for reinforced-concrete design was a series of reports by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete. The name “Joint Committee” reflects the fact that no one group was responsible for the specification. The contributors varied over time but generally included the American Society of Civil Engineers (a professional group founded in 1852), the American Railway Engineering Association (an end-user professional group founded 1899), the American Society for Testing Materials (a standards-setting group founded 1898), the American Concrete Institute (an industry-specific group founded as the National Association of Cement Users in 1904 and renamed in 1913), and the Portland Cement Association (an industry-specific group founded 1902). The ages of the organizations are important because they highlight the greater standardization in the US during the 1900s beginning of reinforced-concrete use in buildings compared to the 1870s beginning of steel use in buildings.

The ACI developed and updated its requirements after the Joint Committee had reached a final report in 1921, issuing its “Reinforced Concrete Building Design and Specifications” in 1927 and proposed “Building Regulations for Reinforced Concrete” in 1936. Finally, in 1941, the ACI issued the first version of its full and permanent specification, “Building Regulations for Reinforced Concrete.” The current revision of this document is the standard concrete code referenced by the various state and local building codes (and referenced in the national standard IBC.) The ACI code is usually called by the author group (ACI Committee 318) and revision year, so that the current code is ACI 318-19.

One of the most basic questions asked in renovation is the load capacity of an existing floor. Given the changes in beam design from the early Joint Committee specifications to the mid-century ACI code, to the modern ACI code, it cannot be assumed that current codes accurately reflect the strength of old concrete structures. This paper provides an overall analysis of the changes in design calculations over the course of the twentieth century in order to provide a schematic-level baseline for making decisions about current capacity of the existing concrete building stock.

2 CODE REQUIREMENTS FOR REINFORCED CONCRETE DESIGN

2.1 Before National Standards

Because of the decentralized nature of American building control prior to the 1950s, there are two critical sources of information about the designs standards in use: local codes that had statutory control over design and national standards that were sometimes (and with increasing frequency as time went on) used as references in local codes.

By later standards, the New York City Building Code of 1901 was extremely conservative about the stresses allowed in concrete.[1] Compression was limited to 230 psi (1.6 MPa) maximum in compression, while steel was allowed up to 16,000 psi (110 MPa) tension. No guidance was given on design, and concrete was generally treated as an unreinforced form of mass masonry. While there were various work-arounds for designers, such as applying for a variance for a specific building, the situation was not properly addressed until the 1916 revision of the code, which included provisions similar to those of the Joint Committee.[2]

The Chicago code of 1905 was less conservative than the New York code with respect to concrete itself, allowing flexural compression in concrete of 500 psi (3.4 MPa), and concrete shear of 75 psi (0.52 MPa), but only allowed tension in the steel of 1/3 of the yield stress, or 11,000 to 13,000 psi (76 to 90 MPa) for the steels commonly in use at the time.[3]

The national standards were spread via engineering professional journals and then incorporated into the local and state laws regarding building enforcement. It is not a coincidence that, for example, the 1912 Detroit Building Code has a calculation method and allowable stresses that match the 1909 Joint Committee report.[4]

2.2 The Joint Committee Reports

The preliminary Joint Committee reports of 1909 and 1913 contain essentially the same requirements for beam design as the "final" report of 1916.[5, 6, 7] The important differences are in materials specifications, column analysis, and less common forms of design, as the basics of beam design had been worked out earlier. The 1916 Joint Committee report provides a good summary of the state of reinforced-concrete beam design in the US in the 1910s.[7] There are descriptions of a limited number of possible concrete mixes, as well as a table with a list of ultimate compressive strengths based on the aggregate type and on the fine-to-coarse aggregate proportions. The list of allowable stress is quite simple:

- Maximum allowable flexural compressive stress is 32.5% of the concrete strength.
- Allowable shear stress without shear reinforcing is 2% of the concrete strength.
- Allowable stress in steel is 16000 psi (110 MPa).

The elastic modulus for steel (E_s) was taken as 29,000,000 psi (200,000 MPa). The elastic modulus for concrete was typically expressed in terms of the modular ratio n (equal to E_s/E_c). The 1916 report gave n=15 for concrete with a compressive strength between 800 and 2200 psi (5.5 and 15 MPa), which covered the bulk of structural reinforced concrete at that time. It also gave n=40 for concrete 800 psi (5.5 MPa) or weaker, n=12 for concrete between 2200 and 2900 psi (15 and 20 MPa), and n=10 for concrete stronger than 2900 psi (20 MPa).

Using the basic design assumptions for a rectangular section beam, (where d is the depth of the beam from the centroid of the tension reinforcement to the top of the beam, b is the width of the beam, A_s is the steel reinforcement area, and p is the steel ratio A_s/bd) the position of the neutral axis is represented by kd, where k is less than 1 and calculated as

$$k = \sqrt{2pn + (pn)^2} - pn \tag{1}$$

The moment arm between the centroids of the steel and the concrete compression is represented by jd, where j is calculated as

$$j = 1 - \frac{k}{3} \tag{2}$$

The calculated maximum stresses in steel and concrete are

$$f_s = \frac{M}{A_s jd} \tag{3}$$

$$f_c = \frac{2M}{jkbd^2}$$

The 1916 report represented a plateau for reinforced-concrete design in the US, but not for research. The basic provisions of that report were in use for roughly 25 years with only minor changes. The same organizations formed a new Joint Committee in 1919 with the purpose of addressing a gap in the 1916 report: creating a specification to accompany the report's design provisions. The 1921 progress report and the 1924 final report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete repeated the design provisions of the 1916 report, while adding a great deal of information, in specification form, on the preparation and construction of concrete.[8, 9] The areas of design that were of greatest interest in research, and therefore the areas where changes were most likely to occur, were column design and two-way slab design, neither of which affects the topic of this paper. The last Joint Committee was formed in 1930 to review the 1924 Specification and the 1916 report and bring them up to date as necessary. The report of this committee was issued in 1940, and contained the following notation regarding ordinary beam design: "The standard formulas for rectangular and T-beams and standard notation have become so widely recognized that they are omitted from the Report."[10]

2.3 ACI Standards and Codes

Of the organizations in the Joint Committees, only the American Concrete Institute (including its years as the National Association of Cement Users) was dedicated to the issues of professional designers. As a new group, it benefited from collaborating with the better-established ASCE. In 1908 the NACU issued its "Report of the Committee on Laws and Ordinances."[11] No analysis equations were provided and the statement that the stress-strain of concrete "may be assumed (a) As a straight line. (b) As a parabola, with its axis vertical and its vertex on the neutral axis of the beam, girder or slab; or (c) As an empirical curve, with an area one-quarter greater than if it were a straight line, and with its center of gravity at the same height as that of the parabolic area assumed in (b)" suggests that the NACU was not settled on linear-elastic analysis. A safety factor of 4 was used against the steel yield stress, assumed to be 32,000 psi (220 MPa), and the concrete ultimate stress, assumed to be 2000 psi (14 MPa). The modular ratio n was taken as 18; the shear stress for unreinforced 2000-psi concrete was taken as 200 psi (1.4 MPa).

The specification was probably not used very much because it came out only a year before the Joint Committee's first report was issued. The Joint Committee had the weight of better-established organizations behind it and was significantly more conservative. There is no way to be certain now to what extent the NACU 1908 code was used, but designs that conform to its outlier criteria - the high allowable shear stress, for example - do not appear to be common, and the values given in the report do not typically appear in local codes of the era.

The NACU abandoned the 1908 report in its 1910 "Standard Building Regulations for the Use of Reinforced Concrete."[12] This is essentially the 1909 Joint Committee report, which is to be expected since the NACU was participating in the Joint Committee. Linear elastic behavior was assumed, but no equations given. Materials assumptions included 2000 psi (14 MPa) concrete, allowable tension (after a safety factor was applied against yield) of 16,000 psi (110 MPa) for "medium steel" equivalent to structural steel and 20,000 psi (140 MPa) for "high elastic steel." Using those materials, they recommend the use of n=15, maximum flexural compression of 650 psi (4.5 MPa), and maximum concrete "web stresses" (i.e. shear) of 40 psi (0.28 MPa). These values are the same as the Joint Committee.

In 1920, the ACI issued Standard Specifications 23, "Standard Building Regulations for the Use of Reinforced Concrete" which is, in form, a building code for reinforced concrete.[13] The general provisions are the same as the 1916 Joint Committee report except as follows:

- Mix proportions were provided for concrete strengths up to 3000 psi (21 MPa).
- Allowable flexural compression was increased to 37.5% of the compressive strength.
- Allowable tensile stress in steel was 16,000 psi (110 MPa), except 18,000 psi (124 MPa) was used when the yield stress was over 50,000 psi (340 MPa).
- Allowable concrete shear stress with only longitudinal reinforcing was 2% of the compressive strength, but could be increased to 3% if the bars were anchored.

The 1920 ACI standard is more detailed and less conservative than the Joint Committee 1916 report, as the ACI was including newer research into the performance of the material. It should be noted that in dealing with analysis of extant buildings designed under the 1920 standard, anchorage of the reinforcing cannot be assumed and is difficult to check, so the unanchored shear provisions should be used.

The ACI's 1927 "Tentative Building Regulations for the Use of Reinforced Concrete," document E-lA-27T, is an update to the 1920 specification.[14] The extent to which linear-elastic flexural analysis was the standard can be seen in the fact that no equations for analysis were given, but the document states "The customary formulas or their equivalent shall be used." The only change to the specifications for the ordinary beam analysis reviewed here is that the allowable flexural compression was increased to 40% of the concrete compressive strength.

The next code version, in 1936, removed "tentative" from the title but kept "T" in the designation ACI 501-36T.[15] (Committee 501 replaced committee E-1.) The statement on flexural analysis was updated to "The accepted theory of flexure as applied to reinforced concrete shall be applied to all members resisting bending." The provisions for beams were the same except that allowable steel tension was generally increased to 20,000 psi (140 MPa); and the modular ration was defined by using 30,000,000 psi (210,000 MPa) as the elastic modulus for steel, and 1000 times the ultimate compressive stress for concrete.

Finally, in 1941, the ACI code, "Building Regulations for Reinforced Concrete (ACI 318-41)" reached its current designation (committee 501 was renamed to 318); all later codes have been based on this version.[16] The only change relative to beam analysis was that the allowable flexural compression was increased to 45% of the concrete compressive strength. The 1947 revision of the code had no changes relative to beam analysis.[17]

The 1951 ACI code revision 318-51 was the same for beam design except that the allowable concrete shear stress was increased to 3% of the compressive strength and the provisions regarding the effect of anchoring longitudinal reinforcing were removed.[18]

The 1956 ACI code had no changes in basic beam provisions but introduced an appendix "Abstract of Report on Ultimate Strength Design."[19] This served more to spur discussion than as a usable option, as it lacked the detail of the main code. The analysis included:

- Compressive stress distribution could take any geometry "which results in ultimate strength in reasonable agreement with comprehensive tests."
- The load factors were 1.2 for dead load and 2.4 for live load; there were no resistance factors stated, so they were effectively 1.0.
- The maximum compressive stress (f'_c) for analysis was 0.85 times the concrete strength.
- The maximum tension (f_y) to be used in analysis was 60,000 psi (410 MPa) or the steel yield stress, whichever was smaller.
- The analysis formulas for flexure were

$$M_u = bd^2 f'_c q(1 - 0.59q) \tag{4}$$

$$q = p\frac{f_y}{f'_c}$$

$$q \le 0.40\frac{f'_c}{f_y}$$

The 1963 ACI code was the pivotal moment in that it gave equal weight to allowable stress (ASD) and ultimate strength designs (USD), with parallel chapters describing each. Some new provisions applied to both methods:[20]

- The elastic modulus for steel was changed to 29,000,000 psi (200,000 MPa).
- The elastic modulus for concrete was to be calculated using the formula

$$E_c = w^{1.5} 33\sqrt{f_c'} \tag{5}$$

where w was the density of the concrete in lbf per cubic foot and f'_c is expressed in psi. (Note that the ACI formulas typically have hidden embedded units, so that for equation 5 to work, 33 is not simply a numeric constant but carries units of $(psi)^{0.5}/(lbf/ft^3)^{1.5}$.)

The ASD chapters had the same limits on flexural tension and compression as before, but otherwise the shear provisions, with the allowable concrete shear in slabs as

$$f_v = 2\sqrt{f_c'} \tag{6}$$

and the allowable concrete shear in beams as

$$f_v = 1.1\sqrt{f_c'} \tag{7}$$

The USD chapters form the basis of ordinary flexural design per the ACI from this time forwards. The basics of analysis are that the strain in both concrete and steel is proportional to the distance from the cracked neutral axis, and that the maximum allowable compression strain is 0.003. “At ultimate strength, concrete stress is not proportional to strain. The diagram of compressive concrete stress distribution may be assumed to be a rectangle, trapezoid, parabola, or any other shape which results in predictions of ultimate strength in reasonable agreement with the results of comprehensive tests.” The recommended stress distribution is a rectangular stress block of $0.85f'_c$ intensity, extending from the compression side of the section to $a=k_1c$ away, where c is the depth to the neutral axis, $k_1=0.85$ for concrete up to 4000 psi (28 MPa) strength and is reduced for greater strengths. k_1 was later renamed β_1.

- The load factors (γ) were 1.5 for dead load and 1.8 for live load.
- The resistance factors (φ) were 0.90 for flexure and 0.85 for shear.
- The moment formula form 1956 was restated with a simpler form:

$$M_u = \phi[bd^2 f_c' q(1-0.59q)] = \phi\left[A_s f_y\left(d-\frac{a}{2}\right)\right] \tag{8}$$

$$p \le 0.75 p_b = \frac{0.85 k_1 f_c'}{f_y}\frac{87{,}000}{87{,}000+f_y}$$

where p_b is the steel ratio that gives the balanced condition of simultaneous steel and concrete ultimate stress.

- This code has the first ACI statement of factored shear:

$$v_c = \phi\left(1.9\sqrt{f_c'} + 2500\frac{p_w Vd}{M}\right) \tag{9}$$

where V and M are the unfactored shear and moment, and p_w is the shear reinforcing ratio, so the second term is zero when considering only concrete shear resistance.

The 1971 ACI code is similar, with a few changes:[21]

- The load factors were reduced to 1.4 for dead load and 1.7 for live load.
- USD was made the base analysis method, with the ASD analysis was reduced to a single section of the "Analysis and Design General Considerations" chapter. This marked the effective end of development of the ASD method by the ACI.
- For normal-weight concrete, a simplified formula was provided for the elastic modulus:

$$E_c = 57{,}000\sqrt{f_c'} \tag{10}$$

No changes were made to these provisions in the 1977, 1983, 1989, 1995, and 1999 ACI code revisions, except that ASD was moved from the main text to an appendix.[22, 23, 24, 25, 26]

The 2002 ACI code was the most recent with changes to these provisions:[27]

- The load factors were reduced to 1.2 for dead load and 1.6 for live load.
- The resistance factor for shear was reduced to 0.75.
- The ASD appendix was removed.

No changes have been made to these provisions in the 2005, 2008, 2011, 2014, and 2019 ACI code revisions.[28, 29, 30, 31, 32]

3 COMPARATIVE ANALYSIS

Since the question at hand is the capacity of ordinary beams, the details of those beams (for example, span length, the ratio of span to depth, or the presence of a slab for T-beam action) only matter if the analysis varies with those details. Similarly, the flexural and shear strength depend on material properties: the compressive strength of the concrete, the tensile strength of the steel, and the elastic moduli of both. In general, the geometry is not important, although extreme cases may have differences: for example, there are shear requirements specific to beams that have an unusually high depth to width ratio. The comparison here uses beams with common geometry, reinforcing, and materials.

For existing buildings, the strength of the concrete is a simple issue, as it can be field tested. In a broad sense, the issue of concrete quality can be treated with visual observation of conditions, although there should be a default assumption that a decades-old building without reported structural problems has concrete of a minimally-acceptable quality. The strength of reinforcing bars can be measured using destructive testing on samples.

In order to keep the analysis focussed, only the provisions for ordinary reinforce-concrete beams and one-way slabs are reviewed, for shear and flexure. The use of T-beams is not considered because the provisions for allowable flange width have hardly changed, so the use of T beams has little or no effect on the comparison of one code to another. The effect of shear reinforcing is not included because (a) slabs almost never include shear reinforcing in ordinary practice, and (b) determining the amount of shear reinforcing in extant beams can require many more probes than determinant the amount of flexural reinforcement because of the pre-seismic-design practice of eliminating shear stirrups/hoops in the center of beams.

The case studied was a beam 12 inches (0.30m) wide and 28 inches (0.71m) deep, reinforced with 3 #8 bars, 1 inch (25mm) in diameter, using 2000 psi (14 MPa) concrete and 40,000 psi (280 MPa) steel. (Such a beam could not be designed using T-beam analysis if, for example, there were slab openings near mid-span.) The beam has a simple span of 20 feet (6.1m) and supports a total load of 1200 pounds per foot (18kN/m). Three cases were examined, with dead load to live load ratios of 2.0, 1.0, and 0.5. The load ratio does not matter in the ASD analyses, where all loads are treated the same, but affects the results when reviewing older USD designs.

It is not possible to directly compare the analyzed capacity using the different codes because the use of load factors in USD changes the required strength from that used in ASD. Therefore the code-to-code comparison is made using the ratio of demand (applied shears and moments for the example beam using load factors of 1.0 for ASD and those required by code for USD) to capacity (the calculated allowable shears and moments).

The same beam was analyzed under each successive code, and the demand to capacity ratio determined for that code. Since the example beam was chosen to meet the codes, the demand/capacity ratio is less than 1.0 in almost cases, ranging from 0.98 for shear (Joint Committee) to 0.46 for flexure with the 2.0 dead/live load ratio (current ACI code). The only exception is moment under the 1908 NACU code, which has a ratio of 1.72 because of the safety factor of 4 used on the steel. Table 1 shows the demand/capacity ratios, comparing the required moments and shears (M_{reqd} and V_{reqd}) to those provided in design (M_{des} and V_{des}).

Table 1: Demand/Capacity ratios for the same beam under different codes.

	Joint Comm 1909 to 1940	NACU 1908	NACU 1910	ACI 1920	ACI 1927	ACI 1936	ACI 1941, 1947	ACI 1951, ACI 1956 ASD	ACI 1956 USD	ACI 1963 ASD	ACI 1963 USD	ACI 1971 to 1999	ACI 2002 to 2019
DL/LL=2 M_{reqd}/M_{des}	0.86	1.72	0.86	0.85	0.76	0.70	0.68	0.68	0.49	0.68	0.56	0.52	0.46
DL/LL=2 V_{reqd}/V_{des}	0.98	0.20	0.98	0.98	0.98	0.98	0.98	0.65	0.65	0.73	0.86	0.76	0.77
DL/LL=1 M_{reqd}/M_{des}	0.86	1.72	0.86	0.85	0.76	0.70	0.68	0.68	0.59	0.68	0.58	0.55	0.50
DL/LL=1 V_{reqd}/V_{des}	0.98	0.20	0.98	0.98	0.98	0.98	0.98	0.65	0.65	0.73	0.90	0.80	0.83
DL/LL=0.5 M_{reqd}/M_{des}	0.86	1.72	0.86	0.85	0.76	0.70	0.68	0.68	0.69	0.68	0.61	0.58	0.53
DL/LL=0.5 V_{reqd}/V_{des}	0.98	0.20	0.98	0.98	0.98	0.98	0.98	0.65	0.65	0.73	0.94	0.84	0.89

The demand/capacity ratios for each code were then compared to the current code. The moment capacity ratio (original demand/capacity ratio divided by the current demand/ capacity ratio) is always over 1.0 because the calculated moment capacity has increased over time. The ASD moment capacity ratios are quite large, with the smallest (1.28) showing a 28% increase in capacity from re-analyzing under the current code. The USD moment capacity ratios are smaller, but with increase of at least 8%, with the smallest increase for the high-live-load case under the 1971-1999 ACI code.

However, the shear capacity ratios vary widely, generally from 0.74 to 1.27. (Again, the 1908 NACU code is an outlier because of its high allowable shear stress.) In short, the 50 percent increase in ASD shear strengths in the 1951 ACI code resulted in unconservative shear designs for the concrete by current standards. The 2002 reduction of the shear resistance factor from 0.85 to 0.75 means that the even 1971-1999 bare-concrete shear designs are slightly unconservative by current standards. Table 2 shows the shear and moment capacity ratios, comparing original analyses (M_{orig} and V_{orig}) to current (M_{cur} and V_{cur}).

Table 2: Ratio of original to current Demand/Capacity ratios.

	Joint Comm 1909 to 1940	NACU 1908	NACU 1910	ACI 1920	ACI 1927	ACI 1936	ACI 1941, 1947	ACI 1951, ACI 1956 ASD	ACI 1956 USD	ACI 1963 ASD	ACI 1963 USD	ACI 1971 to 1999
DL/LL=2 M_{orig}/M_{cur}	1.86	3.74	1.86	1.85	1.65	1.51	1.48	1.48	1.07	1.48	1.21	1.13
DL/LL=2 V_{orig}/V_{cur}	1.27	0.25	1.27	1.27	1.27	1.27	1.27	0.85	0.85	0.95	1.11	0.99
DL/LL=1 M_{orig}/M_{cur}	1.73	3.47	1.73	1.72	1.53	1.40	1.38	1.38	1.19	1.37	1.18	1.11
DL/LL=1 V_{orig}/V_{cur}	1.18	0.24	1.18	1.18	1.18	1.18	1.18	0.79	0.79	0.88	1.08	0.96
DL/LL=0.5 M_{orig}/M_{cur}	1.61	3.24	1.61	1.60	1.43	1.31	1.28	1.28	1.30	1.28	1.15	1.08
DL/LL=0.5 V_{orig}/V_{cur}	1.10	0.22	1.10	1.10	1.10	1.10	1.10	0.74	0.74	0.82	1.05	0.94

4 CONCLUSIONS

In practice, reuse projects will include analysis of the specific extant beams and slabs. However, an analysis of the type in this paper can be used to give a schematic view of what is to be expected. Existing structure is typically grandfathered against code changes, but if alterations are performed they must meet current code. Therefore knowing that shear capacity is more likely to be a problem in design than moment, the design and investigations can be appropriately focused. Obviously, the capacity of existing shear reinforcing must be a priority, as is looking for changes to the live load and the live-to-dead-load ratio.

There were two significant changes during the evolution in the calculation of moments: from a linear-elastic model of flexure to an ultimate-strength model, and from allowable stress to the use of load and resistance factors. Because both took place at the same time, they are often conflated, but are actually quite different. Load and resistance factors could be used (rather than a fixed safety factor) to obtaining a safety margin with a linear-elastic model; a fixed safety factor could be used with an ultimate-strength model. The change in model more accurately reflects the actual distribution of stress in a reinforced-concrete beam, and also provides a larger moment capacity, all other factors being equal. On the other hand, the change from a fixed safety factor to combined load and resistance factors can be outcome neutral for some dead-load to live-load ratio. This difference can be seen in the results: shear does not benefit from the model change and therefore has no particular pattern over time, while moment has a general increase in capacity because of the model change.

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