In this article, we’ll summarize recent work by ACI Committee 437, Strength Evaluation of Existing Concrete Structures, to provide updated recommendations for load testing as a means of safety verification for existing concrete structures. We’ll also discuss the debate among ACI members regarding proposed changes to load testing practice. More detailed information on these topics will soon be available in a report\(^1\) by ACI Committee 437.

**BACKGROUND**

In 2002, significant revisions were made to ACI 318, “Building Code Requirements for Structural Concrete.”\(^2\) Perhaps the most significant changes involved the modification (in general, reduction) of the load factors to conform to those provided in ASCE 7-98.\(^3\) The strength reduction factors (\(\phi\)-factors) in ACI 318-02 were also reduced to be compatible with the new load factors and produce designs similar to those produced using the prior codes. One exception was that the \(\phi\)-factor for calculating design strength of tension-controlled sections subject to flexure was not changed. The decision to leave the \(\phi\)-factor unchanged for tension-controlled flexural sections was based on historical performance and on more recent studies that indicated this failure mode had developed higher reliability since the time the \(\phi\)-factors had first been established.

As it has been since 1971,\(^4\) the test load currently required for strength evaluation is defined as \(0.85 \times (1.4D + 1.7L)\),\(^5\) where \(D\) is the total dead load and \(L\) is the total live load. So, although the load factors used for design were changed in ACI 318-02, the test load was not changed. In fact, with the reduction in load factors incorporated in ACI 318-02, the test load approached the design load for structures designed in accordance with the new load factors.

In preparing its report, ACI Committee 437 conducted an extensive review of historical load test practice and acceptance criteria in the U.S. and other countries. This review was deemed necessary not only because of the previously mentioned disconnect between historical design load combinations and the required test loads, but also because the procedures and acceptance criteria provided in ACI 318-71 were already some 35 years old when they first appeared. The rationale behind the load test protocol, test load magnitude, and deflection acceptance criteria has therefore become unclear to current practitioners.

**HISTORICAL PRACTICE**

The practice of load testing concrete structures in the U.S. began in the 1890s as a means of proof testing newly constructed concrete systems and structures (Fig. 1). Load testing furthered the development of reinforced...
Concrete by demonstrating the safe load carrying capacity of the numerous complex and proprietary reinforcing systems then being developed in the U.S. and Europe. Even in the early 1900s, however, it was understood that the load testing procedures didn’t provide a great deal of scientific information. Although a test served its purpose by proving to architects and owners that a given building would sustain the loads for which it had been designed, a test did not indicate what percentage of the strength of the structure had been reached during the test.

It became common practice in the U.S. to test concrete structures to a test load magnitude of $2.0L + 1.0D$. This practice was reflected in the 1920 edition of ACI’s “Regulations for the Use of Reinforced Concrete.” These practices clearly established the concept of a test load to demonstrate that a structure had a proven ability to support loads exceeding service loads by a safe margin without ill effect.

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The use of twice the live load in defining the test load magnitude remained a part of all but one of the ACI codes from 1920 up to 1956. What did vary during that period was the percentage of in-place dead load that was also added to the test load, and this changed with almost every new edition. Unfortunately, no written record has been found for the rationale behind the various changes that have occurred over the years.

With the redefinition in 1971 of the test load magnitude as 85% of required strength, the concept of load testing as proof testing became less clear. The very title of Chapter 20 in ACI 318, “Strength Evaluation of Existing Structures,” suggests the possibility of load tests providing proof of strength and even compliance with code requirements. We think the purpose of load testing needs to be clarified, and, as part of this clarification, load testing should be divided into three categories:

- Proof testing to show that a structure can resist design loads with an adequate factor of safety against failure;
- Proof testing to show that a structure can resist working design loads in a serviceable fashion with deflections and cracking within limits considered acceptable by ACI 318; and
- Testing to determine the ultimate strength of a structural element or system, either in the field or in a laboratory setting.

The committee’s review of U.S. and international practice revealed that there hasn’t been a lasting consensus as to what constitutes an appropriate test load to achieve proof loading that will establish safety. Since 1951, the test load in the U.S. has been gradually reduced. With the introduction of strength design concurrent with the development of prestressed concrete design in the U.S., it was understood that newer structures would be more flexible and have lower strength reserves than structures designed using allowable stress methods. The current test load magnitude in the U.S. remains among the highest with respect to international practice.

**SELECTION OF LOAD FACTORS**

Following the change of load factors in ACI 318-02, members of ACI Committees 318 and 437 agreed that an arbitrary reduction in the test load to follow the new load factors was simply not acceptable practice. Because ACI Committee 318 didn’t consider the changes in ACI 318-02 to comprise a reduction in safety, however, the changes alone didn’t provide justification for a reduction in the safety proven by load testing. Further, the current practice had been used extensively for decades and, through experience, been shown to establish satisfactory levels of safety.

To add clarity to the intent of load testing, we’re recommending that the test load magnitude be redefined to explicitly signify its function as a proof load that verifies safety (rather than strength). The proof load can be defined in terms of those parts of the total load that are variable. Dead load is therefore separated into two categories, dead load due to self-weight $D_w$ and dead load due to weight of construction and other building materials $D_s$. Here, $D_s$ includes the weights of finishes, cladding, partitions, and fixed landscaping elements such as planters. $D_w$ should be based on the as-built dimensions of the portions of the structure to be tested or dimensions of the structural elements that are considered to be representative of the as-built structure, if different. Because...
When all suspect portions of a structure are to be load tested, or when the members to be tested are indeterminate and the suspect flaw or weakness is controlled by flexural tension, the TLM (including dead load already in place) shall not be less than the larger value found using Eq. (1), (2), or (3)

\[
TLM = 1.2(D_w + D_s) \quad (1)
\]

\[
TLM = 1.0D_w + 1.1D_s + 1.4L + 0.4(L, \text{ or } S \text{ or } R) \quad (2)
\]

\[
TLM = 1.0D_w + 1.1D_s + 1.4L \quad (3)
\]

where \(L\) equals the roof live load, \(S\) equals the snow load, \(R\) equals the rain load, and the other terms are as previously defined. When only part of suspect portions of a structure is to be load tested, and members to be tested are indeterminate, the TLM (including dead load already in place) shall not be less than the larger value found using Eq. (4), (5), or (6)

\[
TLM = 1.3(D_w + D_s) \quad (4)
\]

\[
TLM = 1.0D_w + 1.1D_s + 1.6L + 0.5(L, \text{ or } S \text{ or } R) \quad (5)
\]

\[
TLM = 1.0D_w + 1.1D_s + 1.6L \quad (6)
\]

In Eq. (2), the coefficient applied to \(L\) may be reduced in accordance with the requirements of the applicable building code. If impact factors have been applied to the live load in design of the structure, then the same impact factor should be applied to \(L\) in Eq. (2), (3), (5) and (6).

The total dead load shall include all superimposed dead loads, \(D_s\), considered in design or considered by the engineer or building official to be relevant to the proposed load test. Where superimposed dead loads represent a significant portion of the total service loads, are not already in place on the structure, or may not be of controllable intensity, a factor greater than 1.1 shall be considered for \(D_s\).

Because cost considerations will make it rare for all suspect portions of a structure to be tested, the basic test load for most indeterminate structures will be governed by Eq. (4), (5), and (6). We’ve examined this test load for a variety of structural systems and intended uses.

The goal of ACI Committee 437 has been to obtain a consistent proof load ratio—the difference between the test and unfactored dead loads divided by the unfactored live load—across a spectrum of live and dead loads and a variety of structural systems. Using the proposed equations, the proof load ratio will be very close to 1.60 unless the superimposed dead loads are very large. For most cases, the TLM defined by Eq. (1) through (6) will be within 4% of the test load currently defined in Chapter 20 of ACI 318-05. The greatest departure of the TLM from the current code requirements occurs when the live to dead load ratio is small—for these cases, Eq. (1) and (4) are designed to ensure an adequate lower bound to the test load.

**LOAD TEST PROTOCOL**

In over 100 years of U.S. practice, the test load has been applied in increments of 25% of the full test load, with deflection measurements made at each load increment. After the full test load is in place, deflection measurements are taken immediately. At the end of a 24-hour holding period, another set of deflection measurements are taken, and the test load is then removed. The deflection recovery is measured after a 24-hour rest period. Test loads are generally uniformly distributed to match the load distribution adopted for design. The purpose of the 24-hour holding period is to permit at least some time-dependent effects, such as creep and load redistribution within the structural system, to occur.

The cost and time required to perform the standard monotonic 24-hour load test (Fig. 2) has driven a search for alternative methods, and a different protocol has been
introduced to the U.S. during the past 10 years. This protocol, known as the cyclic load test method, involves the application of cycles of loading and unloading, using hydraulic jacks to produce increasing magnitude with successive cycles (Fig. 3). The method has primarily been used to evaluate changes in strength and stiffness produced by various retrofitting techniques—ACI 437R-03 provides a detailed discussion.

**ACCEPTANCE CRITERIA**

The load testing provisions of Chapter 20 in ACI 318-05 define acceptance criteria for interpreting the results of the 24-hour monotonic load test. The member or structure is evaluated based on two different sets of acceptance criteria: first, there must be no visual evidence of pending failure, such as spalling or crushing of compression zone concrete; and second, measured deflections must satisfy one of the following two limits:

\[
\Delta_1 = \frac{l_t^2}{20,000h} \quad (7)
\]

\[
\Delta_r \leq \frac{\Delta_1}{4} \quad (8)
\]

where \(\Delta_1\) is the maximum displacement measured during application of the test load relative to the initial position measured no more than 1 hour prior to the application of the test load, \(l_t\) is the span of the member under the test load, \(h\) is the overall thickness or height of the member being tested, and \(\Delta_r\) is the residual deflection—the maximum displacement measured 24 hours after removal of the test load relative to the initial position.

If measured maximum and residual deflections don’t satisfy these equations, it’s permitted to repeat the load test no sooner than 72 hours after the initial load is removed. During the second cycle of load testing, the deflection recovery must satisfy the following limit:

\[
\Delta_2 \leq \frac{\Delta_1}{5} \quad (9)
\]

where \(\Delta_2\) is the maximum displacement measured during application of the second test load relative to the position prior to application of the second test load and \(\Delta_r\) is the displacement measured 24 hours after removal of the second test load relative to the position prior to application of the second test load. The basic form of Eq. (7) has been in the ACI code since 1936, but its origins can be traced to around 1906. It was derived considering allowable stress design criteria for low strength concrete materials and simple span structural members, and thus has no direct application to evaluating load tests for higher strength concrete in statically indeterminate configurations. While the commentary of ACI 318 explains that deflection recovery after removal of the test load is to be used to determine whether the strength of the structure is satisfactory, we believe that many engineers view Eq. (7) as the primary criterion, and deflection recovery limits of Eq. (8) and (9) as fallback provisions.

Chapter 9 of ACI 318-05 includes a number of provisions for control of deflections. Table 9.5(b) provides maximum permissible computed deflections for roof and floor construction. Various limits are provided depending on

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**Fig. 2:** Load test in progress using water in barrels. The traditional 24-hour monotonic load test can be expensive and inconvenient to conduct on in-service buildings.

**Fig. 3:** Load steps and cycles for the cyclic load test protocol.
whether the element supports or is attached to nonstructural elements likely to be damaged by large deflections. These limits are expressed as a fraction of the design span \( l \) (for example \( l/180, l/360, \) and \( l/480 \)).

We believe that if the acceptance criteria for load testing under both service load and full test load conditions could be correlated with the maximum permissible deflection limits in Chapter 9, the acceptance criteria of Chapter 20 would be considerably less confusing to practitioners. The performance of the structure under the test load would be compared with the limits on deflection prescribed for design. We recommend that new deflection acceptance criteria be developed based on the following principles:

- Maximum deflection under service loads should be compared to calculated maximum deflections or code-defined deflection limits for serviceability;
- Maximum deflection under full test load should be compared to calculated maximum deflection at that load level;
- If deflections exceed these two maximums, recovery of deflection after removal of load should be considered; and
- The linearity of deflection response during loading and unloading should fall within a specified limit.

Visual signs of impending failure, such as crushing or excessive cracking should also be considered, although these criteria are more qualitative. We also recommend an upper limit to absolute measured deflection that, if exceeded, would rule out retesting or the option of using deflection recovery as an acceptance criterion. The current recommendation of ACI Committee 437 is that this upper limit be set at \( l/180 \). Recovery of 75% of the maximum deflection should be maintained as a minimum requirement. There is discussion within ACI Committee 437 regarding whether the deflection recovery during a second load test should be greater than the current limit of 80%. Based on research into deflection recovery, a lower limit of 90% has been suggested but not incorporated into the report at this time. Finally, the cyclic load test procedure brings with it the possibility of defining a different set of acceptance criteria—additional information on proposed criteria are contained in ACI 437R-03.8

**CURRENT DEBATE**

A number of key questions remain to be answered. First, we’ve said that the goal of load testing is to prove safety of a structure, rather than determine its strength, but can we define a rational test load that will establish safety? Although the test load defined in ACI 318-05 is time-honored and has been shown through experience to produce acceptable results, it’s still essentially an arbitrary definition, has been subject to numerous changes in the past, and will certainly be changed again.

Over the last 2 years, ACI Committees 318 and 347 have been sharing opinions on this issue. Although there are still significant philosophical differences, there has been progress. For example, to be consistent with the load combinations in ACI 318, Chapter 9, ACI Committee 318 plans to redefine the test load in the next edition of the ACI 318 code. The planned ACI 318 requirements, however, will still include a factor greater than 1.0 applied to all dead loads, including self-weight, and the load factors to be applied to live load are also slightly higher than those proposed by ACI Committee 437. As a result, the test load proposed for the next edition of ACI 318 will be higher than the test load defined in ACI 318-05 and could approach full demand, whereas the one recommended by ACI Committee 437 is on the order of 5 to 10% lower that the current test load.

The philosophical difference between the two proposals is made apparent by comparing the resulting proof load ratios. The proof load ratio produced by the changes planned by ACI Committee 318 ranges from 1.6 to 2.4, depending on the live to dead load ratio, whereas the ratio produced by the ACI Committee 437 recommended test load in Eq. (5) typically falls close to 1.6. Because it is the goal of load testing to prove safety of the structure without causing permanent damage to the structure in the process, there is concern among ACI Committee 437 members with the concept of increasing the minimum required test load. Certainly, this concern is reinforced by anecdotal reports of load testing that caused permanent residual cracking and resulted in legal and financial consequences for building owners.

Second, is it appropriate to use a lower test load for determinate elements, regardless of whether all suspect portions of a structure are load tested? In such elements, we must consider the possibility of producing an inelastic response if the test load approaches the design strength too closely. Even so, lowering the test load effectively requires a lower level of safety, even though the element itself may offer no alternate load paths.

Third, should cyclic load testing be considered along with our traditional 24-hour monotonic load test method? Some major concerns relate to the lack of results using the cyclic load test method on a variety of different structural systems incorporating differing amounts of reinforcing or prestressing. We also lack sufficient test cases comparing the results of the two methods on the same structure so that the safety of the cyclic method can be assessed relative to the 24-hour monotonic load test method. Without a broader range of test results, it isn’t possible to evaluate the suitability of the proposed acceptance criteria for the cyclic test method. If we do not begin to use it broadly for load testing, however, that range of information will never be developed. If we combine the two methods with their different acceptance criteria, we must determine which criteria will control.
Fourth, to what extent should crack widths be part of the acceptance criteria at both service and full test load levels? Although crack width criteria make sense from a design standpoint, they typically apply to normal one-way slab and flexural or flexure/shear cracks and may not apply to two-way systems. Actual crack widths in a real structure are highly variable and difficult to measure consistently. Because of these difficulties, the accuracy to which we can predict crack widths is limited. Some members of ACI Committee 437 believe that crack widths should be used simply as a guide for interpreting the performance of the structure during the test rather than being established as hard and fast rules for acceptance.

Finally, are we ready to define new maximum deflection criteria? We believe a change is appropriate. When testing to service load levels, it would seem rational to relate the performance of the structure during the load test to the limits set on deflections for design purposes. If we can agree on a rational relationship between service load and full test load deflections, we think this approach would add clarity for practitioners when interpreting the results. One of the reasons that absolute deflection limits need to be defined in advance of the test is that, while acceptability would ideally be based on actual performance under load being equal to or better than the predicted performance, the reality is that structures are always more complex than the mathematical models we use to predict their performance. In fact, designers often resort to load tests because previous attempts at mathematically predicting performance have failed to produce convincing or meaningful results.

With the long-term goal of providing building designers, contractors, and owners with reliable, economical, and safe protocols, ACI Committee 437 continues to engage in a lively and collegial debate of these questions. We welcome others to comment.

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Selected for reader interest by the editors.

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