Test Load Magnitude and Acceptance Criteria For Strength Evaluation by Means of Load Testing: Current Recommendations of American Concrete Institute Committee 437 - Strength Evaluation

Thomas E. Nehil, P.E.
Nehil•Sivak Consulting Structural Engineers, 414 South Burdick Street, Suite 300, Kalamazoo, Michigan, United States

Dr. Antonio Nanni, P.E.
Center for Infrastructure Engineering Studies, 1870 Miner Circle, 224 ERL, University of Missouri-Rolla, Rolla, Missouri, 65409-0710, United States

Author 3: Filippo Masetti
Center for Infrastructure Engineering Studies, 1870 Miner Circle, 224 ERL, University of Missouri-Rolla, Rolla, Missouri, 65409-0710, United States

INTRODUCTION

During the last year, the American Concrete Institute International (ACI) has been celebrating the 100th Anniversary since its founding as the National Association of Cement Users in 1904. Interestingly, a number of aspects of current load testing practice date back to that same period.

In this paper, we will be reporting on work within ACI that has been taking place over the last three years to update the provisions in the Building Code Requirements for Structural Concrete, ACI 318, with regard to load testing as a means of strength evaluation, or more correctly safety verification, of existing concrete structures. At the time of this writing, the technical committee within ACI charged with strength evaluation of existing structures, Committee 437, has recently completed a draft document containing recommended changes to current practice. The committee presented its draft report at the fall convention of ACI held in Kansas City, Missouri, the second week of November 2005. We will report on the current debate taking place within ACI regarding proposed changes to load testing practice. Note that any opinions expressed herein are those of the authors and not necessarily of Committee 437 or of ACI.

Keywords: acceptance criteria, cyclic load test, deflection, deterioration, load test factors, load test protocol, monotonic load test, reinforced concrete, strength evaluation.

BACKGROUND

ACI incorporates within its organizational structure many technical committees that have produced some 400 documents, including state of the art reports, design and materials use guides, specifications, and standards. One of the most important of these of is the Building Code Requirements for Structural Concrete, ACI 318. Committee 318 reviews and incorporates as it finds appropriate new information and methods developed by other technical committees specializing in particular areas of concrete materials, their design and use.

Since 1971 the primary design method in the ACI 318 Code has been based on the so-called "Ultimate Strength Method." Prior to that time, allowable stress design was the primary method in the ACI Code. The basic design strength requirement can be expressed as:

$$\acute{\varnothing} R_n \geq U \quad (1)$$

that is, design strength must be greater than or equal to required strength, and where Ǿ is the strength reduction factor, always less than 1.0 and a function of the strength mechanism being considered such as flexure, shear, axial compression, and so on; R_n is the nominal strength based on calculations using procedures of the building code for evaluating the strength of a concrete section, and; U is the factored demand. U is defined in Chapter 9 of ACI 318 as the greatest demand resulting from a series of specific load combinations. Different types of service loads are assigned different load factors and these vary from load case to load case, as for example:

$$U = 1.4 D \quad (2)$$

$$U = 1.2 D + 1.6 L \quad (3)$$

where D represents the service, or working, dead loads due to self-weight and fixed partitions and equipment, and L represents the service live loads as defined in the general building code. The above two example equations are simplified versions of the actual equations presented in ACI 318 and are given for illustrative purposes only here.

As noted above, the ultimate strength design method became the primary design method in ACI 318 in 1971. The load factors to be applied to service loads in that code had been developed during the 1960's. The basic load combinations in the 1971 edition of the code typically incorporated a load factor of 1.4 applied to dead loads and 1.7 applied to live loads. In 2002, significant revisions were made in ACI 318 when the load factors were modified, and generally reduced, to be in conformity with the American Society of Civil Engineers standard Minimum Design Loads for Buildings and Other Structures. ASCE 7. This standard was the one referenced by the International Building Code, which became the primary general building code in the United States in the year 2000. Ǿ factors in ACI 318 were also reduced in 2002 to be compatible with the new load factors and thereby produce similar design strengths as the prior codes. One exception to this was that the Ǿ factor to be applied when calculating design strength of flexural members controlled by tension in the reinforcing steel was not changed. The decision to leave the Ǿ factor for tension-controlled flexure unchanged was based on historical performance and on studies that had indicated this failure mode had higher reliability.

The ACI 318 Code contains provisions for strength evaluation of existing concrete structures. Chapter 20 of the code provides for the use of load testing as one means of evaluating existing structures. Since 1971 the test load magnitude for such strength evaluation has been defined as 85 percent of the required strength, that is 0.85 (1.4D + 1.7L). The test load magnitude was not changed in the 2002 edition from previous editions. With the reduction in load factors incorporated in ACI 318-02 (where the designation "-02" indicates the year of publication, i.e. 2002), the test load magnitude now approached the required strength for structures designed in accordance with the new load factors.

The reduction in load factors used for computing required strength without a corresponding reduction in the test load definition resulted in two effects. First, the test load was no longer a constant percentage of the required strength. Secondly, the test load was now in the range of 93 to 98 percent of the required strength for tension-controlled sections, the relative magnitude depending on the ratio of the live loads to dead loads.

The changes in ACI 318-02 required a reexamination of the load test requirements of Chapter 20. ACI Committee 437-Strength Evaluation was requested to review and comment on test load magnitude requirements for ACI 318. In preparing its report, Committee 437 has undertaken an extensive review of historical load test practice and acceptance criteria both in the United States and a number of other countries. The reason for such a review was that strength evaluation practice by means of load testing had not changed within ACI in the over 30 years since the introduction of ACI 318-71. Further, the procedures and acceptance criteria provided in ACI 318-71 were already some 35 years old and the rationale behind the load test protocol, the test load magnitude, and the deflection acceptance criteria had become unclear to current practitioners. Following the review of historical and international practice, the committee developed its recommendations to address three key areas of load testing: the magnitude of the test load; the protocol of load testing, and; the acceptance criteria to be used with the protocols. The following sections will review the current recommendations of the committee.

HISTORICAL U.S. PRACTICE AND INTERNATIONAL PRACTICE FOR LOAD TESTING

Committee 437's report contains an extensive review of the development of load testing practice in the United States. Work done in Israel, Czechoslovakia, Great Britain and other countries during the last century was also reviewed and summarized. Mr. Thomas L. Rewerts is the principle author of that chapter of the report and is presenting a paper on the findings of his research at this Congress. Here we will only provide a brief summary and review the conclusions drawn from the research.

The practice of load testing concrete structures in the United States began in the 1890's as a means of proof testing newly constructed concrete systems and structures. Load testing furthered the development of reinforced concrete in the United States by demonstrating the safe load carrying capacity of the numerous complex and proprietary reinforcing systems being developed in the United States and Europe at that time. Even in the early 1900's it was understood that the load testing procedures being used did not provide a great deal of scientific information and certainly did not indicate what percentage of the ultimate strength of the structure had been reached during the test. Rather, the tests were intended to prove to architects and owners that the building would sustain the loads for which it had been designed.

It became common practice in the United States to test concrete structures to a test load magnitude of twice the live load plus the in-place dead load, that is 2.0L + 1.0D. This practice was reflected in the 1920 edition of the American Concrete Institute's Regulations for the Use of Reinforced Concrete. These practices clearly established the concept of the test load as demonstrating that a structure had a proven ability to support loads exceeding service loads by a safe margin without ill effect. Thus at full service loads it was demonstrated there would be no imminent danger of collapse.

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 to be added to the test load, and this changed with almost every new edition. Unfortunately no written record has been found as to the rationale for the various changes in test load magnitude definition over the years.

With the redefinition in 1971 of the test load magnitude as 85 percent of required strength, the concept of load testing as proof testing became less clear and, in the opinion of a number of Committee 437 members, over the last 30 years the purpose of load testing has become blurred in the minds of practitioners. The very title of Chapter 20 in ACI 318, "Strength Evaluation of Existing Structures", suggests the possibility of load testing as providing proof of strength and even possibly compliance with code requirements. Committee 437 is of the opinion that the purpose of load testing needs to be clarified and can be divided into three categories:

A. proof testing to show that a structure can safely resist intended design loads with an adequate factor of safety against failure;

B. 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 ;

C. testing to destruction to show the ultimate capacity of a structural element or system, either in the field or in a laboratory setting.

The review of U.S. practice and international practice reveals that there has been no lasting consensus in the United States or worldwide as to what constitutes an appropriate test load magnitude to achieve proof loading that will establish safety. Load factors to be applied to service dead load and live loads have varied considerably in the U.S. and in Europe.

Since 1951, in the U.S., the magnitude of the test load has been gradually reduced. With the introduction of ultimate strength design concurrent with the development of prestressed concrete design in the United States, it was understood that the newer structures would be more flexible and have a lower strength reserve than structures designed by previous allowable stress methods. The current test load magnitude in the United States is, in fact, at an all-time low, although with respect to international practice it remains among the highest.

Also as a result of its review of historical practice, Committee 437 confirmed that current deflection limits defined in ACI 318 to establish satisfactory performance did not bear any engineering relationship to the magnitude of the test load, and the committee is of the opinion that acceptance criteria need to be updated to provide more meaningful criteria. We will discuss this further in the section on acceptance criteria. Here again, since Mr. Rewerts will be reporting in depth on the historical development of a current acceptance criteria in the United States, we refer you to his paper and to the ACI 437 report for further information.

SELECTION OF LOAD FACTORS FOR DEFINING LOAD TEST MAGNITUDE

Following the change of load factors used to define required strength in ACI 318-02, it was felt by members of both Committees 318 and 437 that an arbitrary reduction in the test load magnitude to follow the new load factors was simply not acceptable practice. Changes to the current provisions require solid, preferably scientific justification. Further, Committee 318 was of the opinion that changes in ACI 318-02 did not represent a reduction in safety of structures designed using the new code provisions, therefore those changes did not constitute a reason for a reduction in the safety to be proven by load testing. Further, the current practice has been used extensively for decades and through experience been shown to establish satisfactory levels of safety. Anecdotal reports suggest that no structure that has passed the load test of ACI 318 has subsequently failed (or at least not those portions of the structure that were tested, unless later changes took place a result of deterioration, etc).

In its current report, Committee 437 recommends that the test load magnitude be redefined to make more explicit and apparent the function of load testing as proof loading, that is safety verification rather than strength verification. As discussed previously, proof loading has historically been the purpose of load testing.

The report equates the test load magnitude with a proof load, that is a load to prove safe performance. The report defines the proof load ratio as the test load magnitude minus the dead loads divided by the service live load. The proof load, that is test load magnitude, and proof load ratio thus are defined in terms of service loads and not in terms of required, or ultimate, strength. The proof load ratio makes explicit the factor of safety of the test load over service live load and in so doing adds clarity to the intent of load testing. The goal of the committee has been to define a test load magnitude that provides as consistent a proof load ratio as possible across a spectrum of varying service live to service dead load ratios, that is a variety of different structural systems and intended uses.

The test load magnitude, then, should be a proof load applied to the structure to prove a safe margin of satisfactory performance beyond code required service live and dead loads. The committee proposes that the proof load be defined in terms of those parts of the total load a structure will likely be subjected to that are variable. Therefore in defining the proof load, the committee has separated the components of dead load that do not vary from those that do. Dead load is 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). This latter category includes weights of finishes, cladding, partitions, and fixed landscaping elements. Dead loads due to self-weight should be based on the as-constructed dimensions of those 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. Since this is a known and existing load, there is no need to apply a factor greater than unity to this self-weight when defining the test load as a proof load.

Superimposed dead loads on the other hand represent a variable that may change over time depending on the owner's use of the facility, and construction and maintenance means and methods. A factor greater than 1.0 is suggested for superimposed dead load. The actual factor to be used will depend on the degree of variability anticipated by the engineer defining the load test or by the building official. Committee 437 recommends a factor of 1.1 be applied to superimposed dead load when defining the proof load.

The factor to be applied to the service live load for defining the proof load varies depending on whether all suspect areas of a structure are to be tested or whether only representative portions of the suspect areas are to be tested. Committee 437 recommends reinstituting a format that dates back to the committee report issued in 1967 wherein two different test load magnitudes were provided. When all suspect portions of the structure are to be tested, a lower load factor is considered appropriate since a greater degree of reliability will be achieved by testing all suspect areas. On the other hand, if only portions of the suspect areas are to be tested, a higher test load is recommended to improved the level of confidence that significant flaws or weaknesses in the design, construction, or current condition of the structure are made evident by the load test.

An exception to the above guidelines is made for statically determinate elements to be tested, for example cantilevers or simple span elements where there is the possibility of producing an inelastic response in the element if the test load approaches the design strength too closely. The lower test load magnitude is recommended for testing statically indeterminate structures.

The definition of the test load magnitude in Committee 437's report is as follows. The equations are proposed to be consistent with the load combinations of Chapter 9 in ACI 318.

> Load intensity – When all suspect portions of a structure are to be load tested, or when the elements to be tested are determinate and the suspect flaw or weakness is controlled by flexural tension, the test load magnitude, **TLM,** (including dead load already in place) shall not be less than:
>
> $$TLM = 1.2\ (D_w + D_s) \qquad (20\text{-}1)$$
>
> or
>
> $$TLM = 1.0\ D_w + 1.1\ (D_s) + 1.4L + 0.4\ (L_r \text{ or } S \text{ or } R) \qquad (20\text{-}2)$$
>
> or
>
> $$TLM = 1.0\ D_w + 1.1\ D_s + 1.4\ (L_r \text{ or } S \text{ or } R) + 0.9\ L \qquad (20\text{-}3)$$
>
> When only part of suspect portions of a structure is to be load tested and elements to be tested are indeterminate, the test load magnitude, **TLM,** (including dead load already in place) shall not be less than:
>
> $$TLM = 1.3\ (D_w + D_s) \qquad (20\text{-}4)$$
>
> or
>
> $$TLM = 1.0\ D_w + 1.1\ (D_s) + 1.6L + 0.5\ (L_r \text{ or } S \text{ or } R) \qquad (20\text{-}5)$$
>
> or
>
> $$TLM = 1.0\ D_w + 1.1\ D_s + 1.6\ (L_r \text{ or } S \text{ or } R) + 1.0\ L \qquad (20\text{-}6)$$
>
> It shall be permitted to reduce the live load **L** in accordance with the requirements of the applicable general building code. If impact factors have been used for the live load in design of the structure, then the same impact factor should be included in the above equations.
>
> The total dead load shall include all superimposed dead loads, $\mathbf{D_s}$, 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, and/or may not be of controllable intensity, a factor greater than 1.1 shall be considered for the superimposed dead load in the above equations for calculating the test load magnitude.

In the above, L_r represents roof live loads, S represents snow loads, and R represents rain loads.

Practically speaking, since all suspect portions of a structure are rarely tested due to cost considerations, the latter equations become the basic test load magnitude for indeterminate structures. Committee 437 has examined the effect this definition of the test load magnitude will have on the test load for a variety of structural systems and intended uses. Unless the superimposed dead loads are very large, such as could occur for example on a supported, landscaped plaza, the proof load ratio will turn out to be very close to 1.60. This redefined test load magnitude will in fact for most cases range within $\pm$ 4 percent of the current load intensity defined in Chapter 20 of ACI 318-05. The new test load magnitude will depart from the current code requirements most when the live to dead load ratio is small. To insure that an adequate lower bound to the test load magnitude is provided for structures with small live to dead load ratios, the committee report defines the minimum test load of TLM = 1.3 D for indeterminate structures when only portions of the suspect areas will be tested, as shown above in Equation (20-4).

LOAD TEST PROTOCOL

For the past 100 plus years, the procedure for applying the test load in U.S. practice has been a single monotonic load cycle with the test load being applied in increments of 25 percent up to the full test load. At each increment, response or deflection measurements are to be made. After the full load is in place, deflection measurements are to be taken immediately and at the end of a further 24 hour holding period. Following the last set of measurements, the test load is removed and the deflection recovery measured after another 24 hour rest period. Test loads are applied generally in a uniform distribution 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 and, if they are significant, to become apparent and measurable.

During the last 10 years, a different protocol has been introduced to the United States and has been used to evaluate changes in strength and stiffness produced by various retrofitting techniques, most often employing fiber-reinforced polymer composites. The cost and time required to perform the standard monotonic 24 hour load test has driven the search for alternative methods for assessing structural performance under load. This new protocol is currently know as the Cyclic Load Test Method. In this method the loads are applied in loading-unloading cycles of increasing magnitude using hydraulic jacks. Dr. Antonio Nanni is presenting a paper on this method and its applications at this Congress. We refer you to his presentation for additional information on this protocol.

SELECTION OF ACCEPTANCE CRITERIA

The load testing provisions of Chapter 20 in ACI 318 define acceptance criteria for interpreting the results of the 24 hour monotonic load test. The evaluation of the member or structure is based on two different set of acceptance criteria to certify whether or not the load test has been passed: first, a set of visual parameters such as no spalling or crushing of compression zone concrete or other evidence of pending failure being evident; second, the measured maximum deflections must satisfy one of the two following equations:

$$\Delta_1 \le l_t^2/20{,}000h \qquad (4)$$

$$\Delta_r \le \Delta_1/4 \qquad (5)$$

In the above, Δ_1 represents the maximum deflection after full test load has been applied, l_t represents the span of the member under the test load, h is the overall thickness or height of the member whose deflections are being measured and Δ_r is the residual difference that is the difference between the maximum deflection reading after applying full test load and the final reading taken 24 hours after removal of the test load.

If measured maximum and residual defections do not satisfy the above equations, then it is permitted to repeat the load test. During the second cycle of load testing the deflection recovery must satisfy the following:

$$\Delta_r \le \Delta_2/5 \qquad (6)$$

where Δ_2 is the maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test.

The basic form of the maximum deflection equation has been in the ACI Code since 1936, and the origins of this equation date back some 20 to 30 years prior to that. The fact is, this expression has been with us so long that current practitioners had lost track of its origins or how it had been derived. As discussed in Mr. Rewerts's paper being presented at this Congress, this equation 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 materials in statically indeterminate configurations.

Chapter 9 of the ACI 318 code not only provides load factors, Ǿ factors, and load combinations for defining design strength and required strength, but also includes a number of provisions for control of deflections. In Chapter 9 a table is presented that provides maximum permissible computed deflections for roof and floor construction. Various limits are provided depending on whether the element supports or is attached to non-structural elements likely to be damaged by large deflections. Typically these limits are expressed as a fraction of the design span, for example $l/180$, $l/360$, and $l/480$. Committee 437 is of the opinion that if the acceptance criteria for load testing under both service load conditions and full test load conditions could be correlated with the maximum permissible deflection limits prescribed in Chapter 9, then the acceptance criteria of Chapter 20 would be considerably less confusing to practitioners. The performance of the structure under the test would be compared with the limits on deflection prescribed for design.

The committee recommends that new deflection acceptance criteria be developed based on the following principles:

A. maximum deflection under service loads should be compared with calculated theoretical maximum deflections or against code-defined deflection limits for serviceably;

B. maximum deflection under full test load should be measured against calculated theoretical maximum deflection at that load level;

C. if deflections exceed the above maximums then recovery of deflection after removal of load should be considered; and,

D. the linearity of deflection response during loading and unloading should fall within a specified limit.

Visual signs of impending failure such as concrete crushing in the compressive zones or excessive concrete cracking are also to be considered although this criterion is more qualitative in nature. The committee also recommends that there be an upper limit to measured absolute deflection which, if exceeded, would rule out the option of using deflection recovery as an acceptance criterion as well as retesting. The current recommendation of the committee is that this upper limit be set at $l_t/180$. Deflection recovery of 75 percent of maximum deflection should be maintained as a minimum level of rebound. There is discussion within the committee regarding whether the deflection recovery during a second load test should be greater than the current limit of 80 percent. Based on the findings of research into deflection recovery, a lower limit to defection recovery of 90 percent has been suggested but not incorporated into the report at this time.

The goal of these recommended changes is to relate the acceptance criterion to the load test magnitude and the anticipated performance of the structure based on calculations. The committee recognizes that a load test is typically undertaken when insufficient information is available to perform a strictly analytical evaluation of the structure. The objective, however is to make the best prediction possible with known information and use that prediction to interpret the experimental results.

The cyclic load test procedure brings with it the possibility of defining a different set of acceptance criteria. We refer you her to Dr. Nanni's paper for additional information on the acceptance criteria being proposed for the cyclic load test protocol.

CURRENT DEBATE

A number of key questions lie before us in the ACI community and we think these questions could have some relevance to the international community as well. First, as we have seen the current test load magnitude is time-honored and through experience has been shown to produce acceptable results, but it is still essentially arbitrary, has been subject to numerous changes in the past, and can be changed again. We have said that the goal of load testing is to prove safety of a structure rather than strength. The question is whether we can rationally define an acceptable test load magnitude that will establish safety, that is, whether we can define safety.

Of course the reason Committee 437 has looked back in U.S. history as well as looked at international practice has been to "pick up the thread," that is find the logic for defining acceptable safety. The lack of consensus on this suggests that there are philosophical rather than engineering issues at work. Ideally one would assume that it would be possible to establish an international consensus on this topic and we believe it remains a challenge to all of us internationally involved in the design and evaluation of concrete structures to come to grips with this issue.

Currently the subcommittee within Committee 318 that deals with Chapter 20 is considering changes to the load test magnitude to be incorporated into the Code that are somewhat different than those currently recommended by Committee 437. Over the last 2 years, the two committees have been sharing progress and opinions and so there is similarity between the proposal being considered by the subcommittee in 318 to the proposal of 437. There is still a significant philosophical difference, however, and the test load magnitude being considered within Committee 318 is higher than that proposed by Committee 437. The primary difference is that the Committee 318 proposed changes still incorporate a factor greater than 1.0 applied to all dead loads including self-weight. The load factors to be applied to live load are also slightly higher. The result is that the test load magnitude being considered by Committee 318 would be slightly higher than the current test load intensity in the ACI 318 Code, where the one recommended by 437 is on the order of 5 to 10 percent lower that the current test load magnitude. The philosophical difference between the two proposals becomes apparent when one examines the proof load ratio produced by the proposed 318 load test magnitude definition for various structural systems and service live loads. The proof load ratio produced by the Committee 318 definition ranges from 1.6 to 2.4 whereas that produced by the current 437 recommended test load magnitude definition as noted previously typically falls close to 1.60

Apart from this difference in proof load ratio, however, the question remains whether there is any rational basis for evaluating the two proposals and determining which one is more appropriate or whether another would be better. Without a rational approach, it boils down to the opinion of one group versus another. Of course these opinions have their basis in a long history of experience, but Committee 437 is desirous of establishing a more scientific basis for proposed changes to the test load magnitude. A number of members of Committee 437 are concerned with the idea of increasing the minimum required test load magnitude in the code since it is the goal of load testing to prove safety of the structure without causing permanent damage to the structure in the process. There has been anecdotal reporting of permanent residual cracking in structures as a result of load testing that has resulted in legal and financial consequences for the building owner.

A question we are still debating within ACI is whether it is appropriate to use a lower test load magnitude for determinate elements whether or not load testing will be undertaken of all suspect portions of a structure. This really becomes an extension of the first issue identified above.

Whether we are ready to consider cyclic load testing along with our traditional 24-hour monotonic load test method is still unresolved. 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. Without a broader range of test results it is not possible to evaluate the suitability of the proposed acceptance criteria for the cyclic test method. On the other hand, if we do not begin to use it broadly for load testing, that range of information will never be developed. So, does it make any sense in the interim to try to combine the two methods? Since the two different test methods have different acceptance criteria, which ones will be considered to control in the event that a cyclic test is combined with a 24 hour holding period at maximum test load? These questions remain open at the time of this writing.

Are we finally ready in ACI to move on from our maximum deflection criteria of $\Delta_1 \leq l_t^2/20{,}000h$? Committee 437 feels that a change to new acceptance criteria is appropriate. Certainly 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. Is it appropriate or possible to relate the maximum deflection at full test loads to the same type of "percentage of span" definitions considering that the full test load may be approaching the required strength? The authors are of the opinion that this approach would lend further clarity for practitioners when interpreting the results of the test if a rational relationship between service load deflections and full test load deflections could be agreed upon.

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. Therefore, considerable energy can be extended attempting to predict deflection performance without necessarily producing a meaningful result. Load tests are commonly resorted to, in fact, when previous attempts at mathematically predicting performance have failed.

There is still discussion taking place within Committee 437 on whether or to what extent crack widths should be part of the acceptance criteria at both service loads levels and full test load levels. Crack width criteria makes sense from a design standpoint, however they typically apply to normal one-way slab and flexural or flexure/shear cracks which may not apply in two-way reinforced slab systems . Actual crack widths in a real structure are highly variable and difficult to measure consistently. The accuracy to which we can predict crack widths is limited, because of these difficulties. members of Committee 437 feel 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.

We in ACI Committee 437 welcome comment and suggestions from the international community as we continue to work to make improvements in our ACI documents.

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