

YOUNG JIRSA'S YEN

by

Mete A. Sozen¹

ABSTRACT

Engineering consensus documents in the U.S. on proportioning of reinforced concrete slabs went through creative and contradictory transformations starting in 1900's until a compromise was reached between mechanics and practice as a result of the Investigation of Multiple-Panel Floor Slabs carried out in Talbot Laboratory of the University of Illinois, Urbana, from 1957 to 1963. This paper traces the work on reinforced concrete slabs in Urbana that started with Arthur Newell Talbot and continued virtually without interruption to the time of Dr. Jirsa's arrival in Urbana to deliver the coup de grace.

Uno no tiene que ser torero para poder criticar a un matador.²

INTRODUCTORY REMARKS

What brought young Jirsa to Urbana, IL, from Nebraska in 1960 was a movement that started in late 19th century. In the year 1872, the engineering college catalog of the then four-year old Industrial University of Illinois brandished the message “*This school is designed to make good practical engineers.*” Five years later, Arthur Newell Talbot of Cortland, Illinois, a small town 60 miles west of Chicago, arrived in Urbana, Illinois, intent on becoming an engineer. He graduated in 1881 with a degree in civil engineering and took a job in railroad construction. In 1885, he returned to Urbana as an Assistant Professor of Engineering and Mathematics. He completed his career as a Professor of Municipal and Sanitary Engineering in 1926 and continued on with his research at Urbana. His tangible contributions to the University of Illinois were the Engineering Experiment Station (1903) and Talbot Laboratory (1930). His intangible but more powerful contribution was the shaping of a way of thinking about structures that lasted over a century and produced a continuous stream of talent such as Duff Abrams, Anton Brandtzaeg, Rex Brown, Hardy Cross, Eivind Hognestad, Vernon Jensen, Ralph Kluge, Arthur Lord, Nathan Newmark,

¹ School of Civil Engineering, Purdue University, West Lafayette, IN.

² One does not have to be a bull fighter to judge one.

Frank Richart, Chester Siess, Willis Slater, Ivan Viest, and Harald Westergaard, a torrent that ultimately produced highly competent and creative structural engineers and researchers the like of James O. Jirsa.

TALBOT

Talbot's unswerving creed was "observation uncluttered by preconception." It was indeed Talbot who fulfilled the promise of the institution to produce "good practical engineers" and initiated the approach to engineering research that survived for over a century. His influence is discernible not only in his own students but also in generations of his students' students. Talbot was still in school when Charles Peirce defined "pragmatism." Stated in engineering terms, the concept would value any theory only in proportion to its success in practice. Conversely, if the observations supported a relationship between input and output, the relationship could be used within the limits of the observations made despite the lack of first principles. One could say that in Talbot's world, as in Alexander Pope's, whatever was, was right. Very early in his career as a civil-engineering researcher, he worked on the problem of resistance to flow through locomotive water columns. He arrived at a simple expression to determine the size of the relief valve and concluded with a curt "This method has had experimental verification." That was all he said in its defense. Talbot maintained that attitude throughout his career. He took an interest in reinforced concrete relatively late in his career, after 1900's. He had kept his connections with the railroad industry. His conception of an engineering experiment station at Urbana emanated from his belief that the railroad industry, among others, needed systematic testing to solve some of their construction problems. In addition to problems related to fatigue of metals, railroad engineers had questions related to behavior of reinforced concrete. The first bulletin, published in 1904, one year after the founding of the University of Illinois Engineering Experiment Station, focused on reinforced concrete beams. It is not wasted print to emphasize the obvious. He could have named the station "The Illinois Center for Engineering Research," but he did not. To him, the proper experiment and its proper observation and reporting without prejudgment were the important goals. In what was essentially a public-relations pamphlet, Breckenridge (Breckenridge, 1906) was explicit about the goal of pragmatism. He wrote, "It is the purpose of the station to carry on investigations along various lines of engineering, and to make studies of problems of importance to professional engineers, and to the manufacturing,

mining, railway, constructional and industrial interests of the state. It is believed that this experimental work will result in contributions of value to engineering science and to the industries of the state, and that the pursuit of such investigations will give inspiration to students and add to the value of the instructional work in the College of Engineering."

In 1906, C. A. P. Turner (Turner, 1909) built the first flat slab creating a strong controversy among structural engineers because Turner used only 25% of the reinforcement demanded by theoretical considerations of the time but still managed to produce serviceable and safe structures. Talbot was interested. He published his first study of the topic in 1916 (Talbot and Slater, 1916), a document that was limited to carefully qualified observations. In 1918, he had the opportunity to test a structure that was slated to be demolished (Talbot and Gonnerman, 1918). Despite the existing professional consensus that the static moment (absolute sum of negative and positive moments in a symmetrically restrained panel with circular columns) would be

$$M_o = 0.09 WL(1 - \frac{2c}{3L}) \quad (1)$$

Talbot expressed it correctly in keeping with Nichols's statement as being approximately (Nichols, 1914)

$$M_o = \frac{1}{8} WL(1 - \frac{2c}{3L}) \quad (2)$$

W : Total uniform load on a slab panel

L : Span in direction considered

c : column diameter

It must have been his pragmatism that a few years later he was willing to accept Eq. 1 for design and also rationalized that the moment demand in isolated footings would be 85% of that based on statics.

WESTERGAARD

Perhaps the most important result of Talbot's interest in slabs was attracting Harald Malcolm Westergaard to come to Urbana as a graduate student. Westergaard had completed his

education in Copenhagen in 1911. Advised by his teacher, A. S. Ostenfeld, to continue for a doctoral degree, Westergaard went to Goettingen and then to Munich and was about to complete his dissertation in 1915 when war in Europe made it impossible for him to continue his studies. This time Ostenfeld advised him to go to Urbana because Ostenfeld thought there was an engineer in Urbana who was doing some work on slabs, a topic in which Westergaard was interested. Westergaard obtained his PhD degree in Urbana and continued his research as a faculty member of the University of Illinois. His cooperation with Slater, another student of Talbot, led to "Moments and Stresses in Slabs," (Westergaard and Slater, 1921). This monumental work settled, at least for all well informed engineers, the controversy resulting from the apparent contradictions between mechanics and observation. In addition to demonstrating the commonalities of flat and two-way slabs in terms of plate theory, the paper was designed masterfully to appeal to the practicing engineer. The dominant discrepancy between measured and calculated reinforcement stresses was laid to rest by a simple plot that deserves mention. The influence of the tensile strength of the concrete on the moment resistance of a reinforced concrete section is by now well known under the misleading name of "tension stiffening" when the physical phenomenon is actually one of "tension softening." At moment demands below the yield moment, under static monotonic load, tensile strength of the concrete helps resist some of the applied moment so that the stress in the reinforcement, inferred from measured strain, may be well below the stress that would be required in the reinforcement to resist the moment if the calculation of stress is based on the assumption of concrete having no tensile strength. If that happened in a simply supported beam, the observer might look for explanation in other factors than in the value of the moment demand. If that happened in a two-way slab, one could be confused by the consideration that load is carried in more than one direction. As Nichols (Nichols, 1914) had demonstrated, there was no credibility to imagining that only part of the load on a slab was carried in one direction but his point of view had not met with unanimous approval of the profession possibly because the complexities of plate theory could be used to confuse. Westergaard and Slater separated the question of measured moment magnitude from theory and made their clarifying statement in a simple plot (Fig. 1) showing the relationship of measured calculated reinforcement stress in beams (where there was no question about the magnitude of moment) to the calculated stress based on the "straight-line formula." Their approach, which was admittedly indirect, convinced more engineers than did Nichols's simple and direct solution for the static moment .

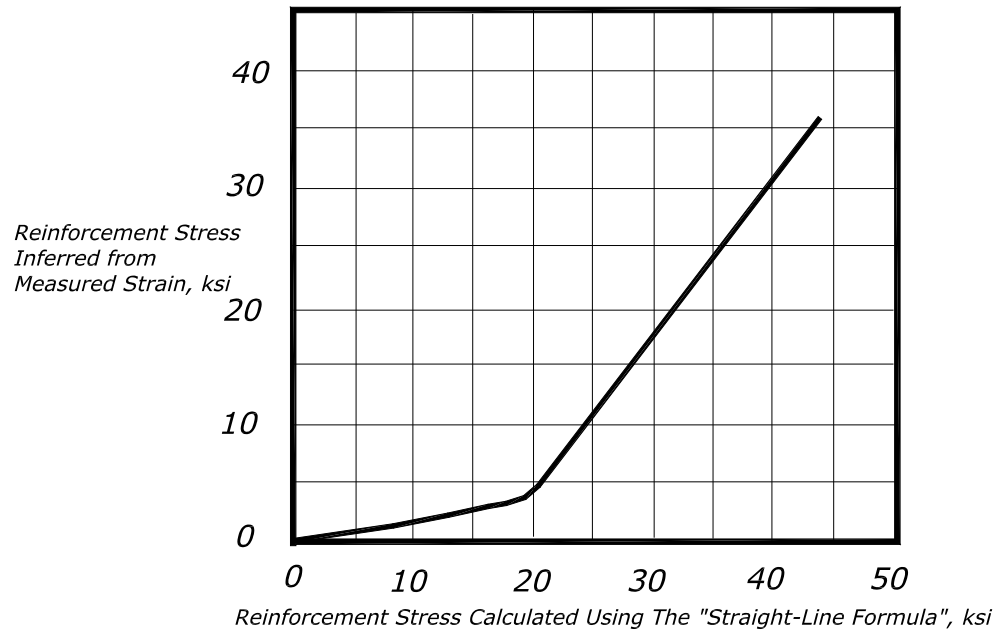


Figure 1 Relationship between "Measured" and Calculated Reinforcement Stress in A Beam

Westergaard and Slater presented a plot of moment distribution in a square interior panel of a flat slab that may be one of the best examples of graphic art in engineering expression. An abbreviated version of their illustration is shown in Fig.2. The useful information packed in their figure matches the celebrated drawing, by another civil engineer (Charles Minard), providing data on Napoleon's 1812 trip to and from Moscow (Tufte, 1984).

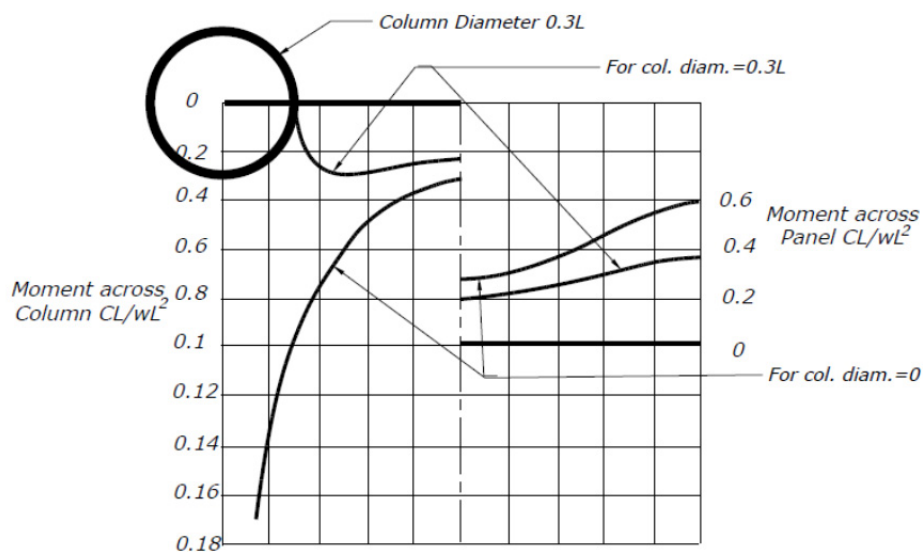


Figure 2 Partial Reproduction of The Moment-Distribution Plot by Westergaard and Slater

Westergaard and Slater went beyond the extensive information given in their version in Fig. 2 After confirming Nichols (Eq. 2) by defining the static moment, M_o , (absolute sum of the positive moment and the mean of the negative moments at each support in one direction) as he did, they went on to propose a very simple table to help designers determine how to distribute the reinforcement in an interior panel (Table 1) .

TABLE 1

	Percent of Static Moment, M_o		
	Column Strip (Width = Panel Width/2)	Middle Strip	Total
Moment at Support	48	17	65
Moment at Mid-Span	21	14	35

CROSS

Hardy Cross, who joined the faculty in Urbana in 1921, wrote that one could proportion two-way slabs simply by considering “.. the limitations imposed by statics upon the total moments, principles of symmetry and asymmetry, and mental pictures of the deflected slab as a means of judging of the variation of the moments along any given section..... The writer finds the idea of distributing fixed-end moments useful in revising his mental pictures of the deflected slab as a means of judging of the variation of the moments when affected by continuity with other slabs or by discontinuities.” It took Ingerslev (Ingerslev, 1921) to state the basic rules of yield-line analysis, Johansen (Johansen, 1931) to generalize the rules, and Hognestad (Hognestad, 1953) to translate their work to English for the majority of the profession in the U.S. to appreciate Cross’s wisdom on proportioning of slabs (Cross, 1929).

NEWMARK

The momentum of interest on analysis of slabs picked up again in Urbana in the 1930's as a result of the cooperative efforts of Westergaard and Cross. For cases where classical closed-form solutions did not apply, Westergaard had used finite-difference solutions, his own and those of others. Nathan M. Newmark, a student of Cross and Westergaard who had joined them on the faculty, was interested in combining the two engineer's approaches to bring about a pragmatic and general approach to design of two-way slabs. Without the computational tools we enjoy today, he was at a disadvantage for obtaining finite-difference solutions to cover a sufficient number of cases. His disadvantage became an advantage because it drove him to think. He projected Cross's brilliant and simple solution for frames to apply to slabs (Newmark, 1938). His work was inspired by Cross's creativity and influenced by Westergaard's talent to simplify the complex. Because the dominant thinking of the time on structural design was focused on bridges³ that were sensitive to pattern loading and because the design criterion was working stress, his focus was on effects of pattern loads on moments in slabs in the linear range of response. Newmark's 1938 opus was influenced by problems related to slabs in bridges. Later, working with his student Chester P. Siess, Newmark shifted his attention to floor slabs in buildings. Their work resulted in a paper (Siess and Newmark, 1948) published in the Journal of the American Concrete Institute with the title "Rational Analysis and Design of Two-Way Concrete Slabs." From the title, it might be inferred that they had deviated from the path set by Talbot, but it was not so. As stated explicitly in their paper, the method they proposed was "not exact since the values of the stiffness and carry-over factors can be determined only approximately." As Talbot would have approved, they recognized the need for tests to calibrate their conclusions and emphasized that they had no directly relevant test results. Their conclusions were based on analytical studies. The paper identified the dependence of stresses in two-way slabs (slabs supported by beams on all four sides) to three factors as well as the magnitudes of the load and the span: (1) relative flexural stiffness of the beam to that of the slab, (2) relative flexural stiffness of the column to the combined flexural stiffness of the slab and the beams, and (3) torsional stiffness of the beams.

³ Because of the dominance of working stresses in design, Siess and Newmark concentrated on the effects of pattern loadings, undeniably a mode of thinking heavily influenced by the preoccupation of the structural-design profession with bridges in the early part of the 20th century

SIESS

The moment distribution method developed by Newmark was extended by Siess and provided the analytical platform for their extensive analysis of the effects of span, load, panel shape, beam flexural stiffness, beam torsional stiffness, and slab flexural stiffness. Despite their explicit consciousness of nonlinear response, of redistribution of moments within the slab and from slab to beam, their quantitative conclusions were influenced by moment-demand changes calculated for linear slabs and beams, that would be caused by pattern loadings. They studied and compared the effects of checkerboard and single-panel loadings with those of uniform loading. They investigated the effects of beam deflections. Although they related safety to effects of pattern loadings because they were operating in the realm of working stress design, that they had not deviated from the Talbot tradition was evident when they stated that a difference as much as 28% between the design moment and the actual could be tolerated.

Newmark and Siess started pushing the bounds set by professional consensus committed to the criticality of pattern loading. Pointing out the discrepancy in design documents leading to a wide difference between the influences of pattern loads in flat slabs and two-way slabs, they showed that typically a monolithic reinforced concrete beam has sufficient torsional stiffness to ameliorate the effect of checkerboard loading and that, to obtain a satisfactory measure of the effect of pattern loading, it was sufficient to consider single-panel loading for most cases.

Even though Newmark and Siess had named their first paper “Rational Analysis and Design of Two-Way Concrete Slabs,” it was clear that their emphasis was on design as they started by stating how approximate their numbers were and named their second paper “Proposed Design Specifications for Two-Way Floor Slabs.” The two papers were quite influential. They convinced the leadership of the structural engineering profession that an organized study was due for the way slabs were considered in design. Indeed, slabs did not constitute the critical elements of the entire structure but they required as much material as the main elements.

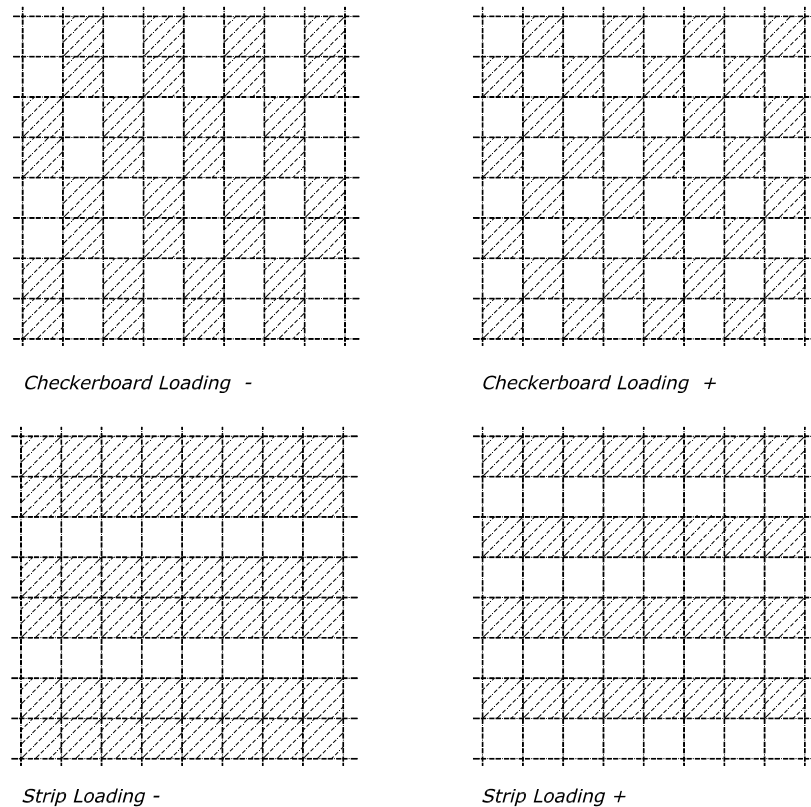


Figure 3 Checkerboard and Strip Loading Patterns Shown for One Direction

In 1957, a broad-front research program led by Newmark and Siess was initiated at Talbot Laboratory of the University of Illinois incorporating both analytical and experimental work (1963). The program had an advisory committee of academic and professional engineers. A total of five multiple-panel slabs (each with nine panels) were tested under gravity loads. The typical test program for each structure comprised a series of strip and checkerboard loadings to the service-load level followed by a test to failure with all nine panels loaded.

JIRSA

Young Jirsa found himself, as a graduate student, in charge of testing the last specimen. Many of the experimental challenges had been solved including the use of automated recording of strain gages (alas on “punch cards”), but the use of reinforcement with a fracture strain below 6% had yet to be understood. Jirsa completed the fabrication and testing of the structure in record time and showed his talent in dealing with the unexpected when he found out that linearity of strain

over the depth of the reinforced concrete section was a fiction maintained traditionally because typical reinforcement used in reinforced concrete structures had a large strain capacity, on the order of a hundred times the yield strain.

After his successful completion of the flat-plate structure with welded wire fabric for flexural reinforcement in the slab, young Jirsa's yen was to simplify if not eliminate the pattern-load problem that tended to complicate design analyses for slab. He was going against professional faith. He had the experimental data from five test structures and the analytical background provided by Newmark and Siess. The challenge was to simplify design. As Vachel Lindsay might have said, young Jirsa was influenced by Talbot, Westergaard, and Cross who stalked the corridors of Talbot Laboratory at midnight. Simplicity was more important than precision. If one was going to be wrong anyway, one might as well be wrong the easy way. Young Jirsa, steeped in the intellectual environment in Urbana, knew that analyses used for design did not necessarily have to predict response. The primary object of design was to obtain a safe, serviceable, and economical structure. Above all, the design method should enable the designer to estimate the result before implementing the method and not hide the outcome. He had read Cross who wrote about slabs, "Analyses indicate certain distributions of reinforcing steel [in slabs], whereas often quite different distributions are just about as effective. No one has pointed this out more clearly than Westergaard (Cross, 1936)."

Young Jirsa did not fail his heritage. First, he observed that the expected variations caused by pattern loads in slab moments at the edges of supporting beams would be in a tolerable range and could be safely ignored in design. By focusing on positive moments, more than half the problem had been eliminated. He invented a different way of sensing the effects of the dominant parameters by combining them in three ratios as follows:

Beam Flexural Stiffness

$$\frac{H}{1+H} \quad (3)$$

$$H_a = \frac{EI_a}{b \frac{Et^3}{12}} \text{ in direction } a$$

$$H_b = \frac{EI_b}{b \frac{Et^3}{12}} \text{ in direction b}$$

Beam Torsional Stiffness

$$\frac{J}{1+J} \quad (4)$$

$$J_a = \frac{GC_a}{a \frac{Et^3}{12}} \text{ in direction a}$$

$$J_b = \frac{GC_b}{b \frac{Et^3}{12}} \text{ in direction b}$$

Column Flexural Stiffness

$$\frac{K}{1+K} \quad (5)$$

$$K = \frac{\frac{\sum k_c EI_{col}}{h}}{\frac{\sum k_s EI_{slab} + k_b EI_{beam}}{a}}$$

a = span length in direction of considered moments

b = span length orthogonal to span a

b₁, b₂ = long and short dimensions of beam section

c = diameter or width of column capital

E = Young's modulus

G = Shear modulus

h = story height

C = R*b₁b₂³ = torsional stiffness of beam with rectangular section in span considered =

R = 0.14 for b₁/b₂=1, R= 0.299 for b₁/b₂=2, R= 0.31 for b₁/b₂ = 10

With those ratios, he could capture and express the effect of the pattern loads in a simple plot

(Fig. 3). For slabs without beams, the parameter H/(1+H) is zero. The critical parameter is

K/(1+K). For practical dimensions of monolithic structures, the parameter K/(1+K) is unlikely to

be below zero resulting in a reasonable upper bound of $a=4/3$ for the ratio of increased moment to that for uniform loading. He reasoned that because $a=4/3$ refers to the entire load applied on panel strips with the other panel strips empty, a is likely not exceed $6/5$ if the live load is not more than three times the selfweight.

For slabs with beams, the strip pattern loading demand is not of primary importance and can be ignored. Considering the effect of pattern loadings on two-way slabs, it is plausible to expect that the values of J and H will be comparable. Inspecting Fig. 4, it is seen that the moment increase, occurring at the intersections of the curves for equal values of $H/(1+H)$ and $J/(1+J)$, is tolerable.

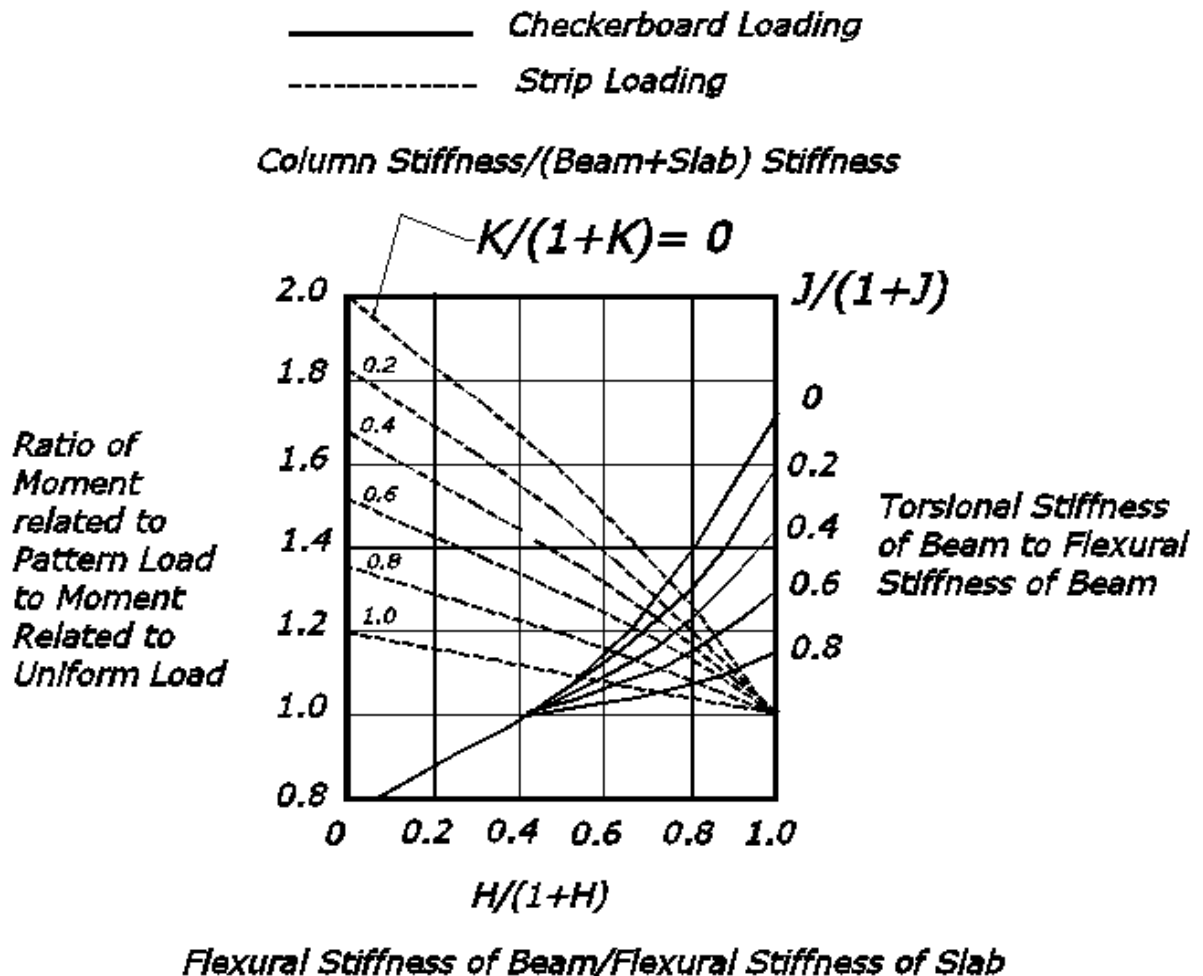


Figure 4 Young Jirsa's Representation of The Pattern-Loading Effects in An Interior Panel

Young Jirsa made two important contributions to the design of floor slabs: (1) He was able to collapse the many dimensions of the pattern-load problem into one figure, not unlike the achievement of Westergaard and Slater in producing Fig. 2 and (2) instead of packaging his solution into a set of formulas to burden the code, he had the self-confidence and the concern for the profession to walk away from the temptation of having, as they call it currently, “updated” the code. He devised a simple table for including the effect of pattern loading on positive moment. This table made its way into the 1971 version of ACI 318. When the ratio of live to dead load was reduced to two in 1995, the table was dropped. Young Jirsa had achieved his goal.

CONCLUDING REMARK

Relegating the effect of pattern loadings to the “back burner” was a very promising beginning for young Jirsa. In retrospect, it appears that lightening the building code is a greater achievement than loading it with procedures that belong in text books. Dr. Jirsa's brilliant career as an eminent engineering teacher and researcher that we celebrate today is not a surprise.

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PULLOUT STRENGTH OF L-BOLT ANCHORS - REVISITING DESIGN EQUATIONS

by

Donald F. Meinheit, Andrew E. N. Osborn, and Mark R. Krueger

Synopsis: Hooked-shaped anchor bolts are a time-honored, commonly used anchor to attach steel column base plates, light poles, wood sill plates, and other structural attachments to concrete foundations. They are typically known as J- or L-bolts and have lost some popularity with the availability of post-installed anchors. Generally, the anchor is a bent bar, smooth or deformed with threads on one end and a hook on the other end. J- or L-bolts are inexpensive and easily manufactured out of straight or coiled bars. When used to attach and anchor steel columns there is often little need to consider uplift tension loading on the anchors. The usual loading is a shear load where strength of the anchor depends on how far the anchors are from the free edge of the concrete. However, for sign posts, hooked anchor bolts are required to carry tension loading due to overturning bending moments from lateral wind loading. This paper reviews the behavior of hooked anchors bolts and proposes design expressions for tension loading.

Keywords: anchors (fasteners), J- and L-bolts, hooked anchors, tension loading (strength), pullout failure, failure modes

ACI Fellow Donald F. Meinheit is a retiree of Wiss, Janney, Elstner Associates, Inc., Chicago, Illinois and now works part time as an Affiliated Consultant. He is a member or past member of several ACI committees: ACI 355, Anchorage to Concrete, ACI 352, Joints and Connection in Monolithic Concrete Structures, ACI 533, Precast Panels, ACI 608, Adhesive Anchor Installer Certification, and ACI 301-F, Specifications Precast Concrete Panels. Mr. Meinheit a past chair of ACI 355 and ACI 533.

PCI Fellow Andrew E. N. Osborn is a Senior Principal at Wiss, Janney, Elstner Associates Inc., New York, NY where he has been employed since 1978. Mr. Osborn engages in a wide range of consulting and research activities. His consulting work deals with the investigation of structural failures while his recent research interest's deal with the bond of prestressing strand to concrete. Mr. Osborn is a past Chair of the PCI Technical Activities Council and PCI Connection Details Committee. He is also a member of the PCI Technical Activities Council, the PCI Research and Development Council, the Prestressing Steel Committee, and the Strand Bond Task Force.

Mark R. Krueger is a Principal and Unit Manager, Wiss, Janney, Elstner Associates, Inc., Detroit, Michigan. Over his more than 25 years of consulting experience, he has conducted and analyzed the results from numerous laboratory and in-situ strength testing programs on a variety of structural elements. He is a member of ACI, serves as the chairman of the Code committee for the Structural Engineers Association of Michigan, and is on the 2012 Michigan Building Code Review Committee.

INTRODUCTION

Information for this paper is a result of a technical inquiry from a fellow Wiss, Janney, Elstner Associates, Inc. (WJE) employee asking the first author what a designer should have known about the tension design of hooked-bar anchors in the early 1980s. Upon responding to the initial question, a more detailed review of the origins of the ACI 318 design equation for tension capacity of hooked-bar anchors was initiated by the first author at which time the coauthors provided historical background information on testing done by WJE.

Background

Hooked-bar type anchors have enjoyed extensive popularity in the past and are still a viable anchor type. However, hooked-bar anchors generally have a tensile strength when embedded in concrete less than that of an equivalent diameter and an equivalent embedment depth of welded headed stud anchors, bolts with a standard head, or threaded rods with heavy-hex nuts embedded at the end of the bolt shank.

Hooked-bar anchors are defined as anchors manufactured from straight or straightened coiled smooth bars and cold bent in either a J- or L-shape at the embedded end and threaded on the other end. Few hooked bar anchors are now bent into a J-shape because it is easier to bend the L-shape and the L-shape requires a shorter length of steel bar to create the hook. Therefore, this paper will primarily discuss the behavior of the L-bolt anchor.

The end of the anchor exposed and protruding out of the concrete, making the attachment, is threaded either by cutting or rolling threads onto the smooth bar. The threaded end of a hooked bar anchor may be induction heated and upset to a larger diameter sufficiently to make the effective cross-sectional area of the upset thread equal to the gross cross-sectional area of the bent L-bolt, thus allowing a nominally smaller diameter bar to be used for the anchor.

Mild steel with yield strengths of 36 ksi (248 MPa) to 55 ksi (379 MPa) are commonly used for L-bolts. The most often cited modern specification for L-bolt anchors is ASTM F1554.¹ The availability of the material, ease of bending, and rolling threads on one end plus the ability to upset the threaded end make the L-bolt anchor a very economical cast-in-place anchor alternative.

The American Institute of Steel Construction (AISC) has for many years included sketches of L-bolt anchors in their design manual.² The AISC Design Manual, however, does not provide design guidance to proportion the L-bolt anchor. Design guidance from AISC for anchors is found in two of the AISC Design Guides.^{3,4}

Scope

The ACI 318⁵ code contains several design equations for calculating the various strengths for anchors embedded in concrete. There is a design equation for the tensile pullout capacity of a hooked anchor bolt whose reference is

relatively obscure.⁶ Because of the technical question posed to the first author, a review of the origins of the ACI code design equation was initiated and an assembled database analyzed.

The authors want to recognize, posthumously, Dr. A. Fattah Shaikh who encouraged the authors' in many ways to study the behavior of anchors in concrete.

GENERAL BEHAVIOR OF HOOKED-BOLT ANCHORS IN TENSION

In tension, there are four very distinct failure modes for a hooked-anchor bolt. The four failure modes are: a. steel fracture, b. concrete cone breakout, c. side-face blowout, and d. pullout of the anchor from the concrete (so-called by ACI Committee 355), often also called a slip out of the concrete failure mode.

Steel failure

A steel failure is when the ultimate load that the anchor can carry is controlled by rupture of the steel used in the anchor. Steel failure also indicates that the embedded depth and distance to a free edge of the concrete are large enough to preclude the other failure modes. Rupture of the steel's cross section at the ultimate load usually depends on the location of the most highly stressed part of the anchor. For L-bolt anchors, steel failure can occur in the threads of the anchor. However, if the anchor has an upset threaded end, the net cross-sectional area of the threaded end may be larger than the area of the shank of the anchor. The photograph in Figure 1 is a case where the anchor shank on the right had a more critical area in the shank than the threads, and therefore, the anchor failed away from the threaded area. A load-displacement curve for a steel failure of an L-bolt anchor is given in Figure 2a.

Steel failure strength is determined simply from Equation 1.

$$N_u = A_{se} (f_{uta}) \quad (\text{Eq. 1})$$

where:

N_u = calculated ultimate load, pounds [lbs] (Newtons [N])

A_{se} = effective cross-sectional area of the anchor, inches² [in²] (millimeters² [mm²])

f_{uta} = ultimate tensile strength of the anchor steel, pounds per square inch [psi] (MegaPascals [MPa])

Concrete breakout failure

Concrete breakout failures occur when the embedment depth, h_{ef} , of the anchor is shallow. This failure mode is also frequently called a concrete cone failure. When the failure load magnitude exceeds the tensile strength of the concrete on a semi-conical surface whose apex is at the embedded end of the anchor the concrete breaks out. Concrete breakout failures considered here also assume that there are no influences of spacing between adjacent anchors and no edge of concrete effects that could decrease the size of the normal breakout cone. Failure in concrete breakout is brittle. A load-displacement curve for a concrete breakout failure of an L-bolt anchor is shown in Figure 2b. Figure 3 is a photograph of an L-bolt with shallow embedment breaking out the concrete before crushing the concrete at the hook bend.

Several concrete breakout failure equations have been proposed over the last 30 years.⁷ The current concrete breakout equation representing cast-in-place anchors is given by Equation 2. This equation represents the average breakout strength of a cast-in-place anchor in uncracked concrete.

$$\begin{aligned} N_b &= 40 (f'_c)^{1/2} (h_{ef})^{3/2} \quad [\text{lbs}] \\ N_b &= 17 (f'_c)^{1/2} (h_{ef})^{3/2} \quad [\text{N}] \end{aligned} \quad (\text{Eq. 2})$$

where:

N_b = average ultimate load, lbs (N)

f'_c = concrete cylinder compressive strength, psi (MPa)

h_{ef} = effective embedment depth, distance down from surface of concrete where the tension load is transferred to the concrete, in. (mm)

Side-face blowout failure

A side-face blowout failure occurs when the anchor is located near a free edge and a radial bursting force develops in the bearing area of the anchor exceeding the lateral, splitting resistance of the concrete. This failure mode will not

be discussed in any more detail other than to mention it is a possible failure mode for an L-bolt anchor when close to an edge of the concrete member.

Anchor pullout failure

The last failure mode is the pullout failure of the L-bolt. Pullout occurs when the embedment is deeper than that to form a concrete breakout but not so deep as to provide enough bond and/or bearing capacity to cause rupture of the anchor steel. In Figure 4, several stages of a pullout failure of an L-bolt are shown. Initially the load in the anchor is transferred to the concrete by adhesive bond of the anchor shank to the concrete, Figure 4a. As the load increases, but is still less than the steel rupture load, the adhesive bond breaks down and the load is transferred to the concrete in bearing at the hook. This state is shown in Figure 4b. It is known that for welded headed studs, a bearing stress under the head of the stud or head of a bolt greater than $12 f_c'$ causes detrimental concrete crushing and permits an increased anchor displacement.⁸ Research on dowels loaded in shear has shown that bearing stresses of about $5 f_c'$ occur where a dowel bears on the concrete at a crack.⁹ In Figure 4c, the concrete is shown being crushed by the high bearing stresses at the hook bend permitting the typically smooth bar to plastically bend and slip around the 90-degree corner, straightening out as the bar is gradually withdrawn from the concrete by the applied tension load. With continued loading, the L-bolt pull outs of the concrete usually causing little spalling at the concrete surface (see Figure 4d). The left hand side of Figure 1 shows a pullout failure of an L-bolt. Figure 5 shows a load-displacement curve for a pullout failure of an L-bolt anchor.

PREDICTION MODELS FOR PULLOUT FAILURES

Historical review

Historically, pullout failure load for post-installed anchors have not been predictable with an equation. For anchor design in tension, the designer is obligated to check the different failure modes and determine which of the failure modes controls the design. Pullout failure load magnitudes for post-installed anchors are typically shown on the qualification sheet from the anchor manufacturer. However, for cast-in-place hooked anchors, the ACI code provides an equation for the pullout strength of hooked-bolt anchors.

Review of WJE 1993 test results—In 1993, Wiss, Janney, Elstner Associates, Inc. conducted a series of 35 in-house tension tests on L-bolts embedded in concrete. Results for these tests and others found in literature were analyzed.¹⁰ Listed in Table 1 is the database of hooked-bolt anchor tests assembled by WJE.¹⁰ Also listed in Table 1 are additional published and unpublished sources of test data.^{11,12} A significant number of the tests were conducted on L-bolts embedded in grouted cells of concrete masonry units. Observations from the WJE tests showed the following features:

- Concrete breakout failures appear to be proportional to the embedment depth instead of the embedment depth squared. This observation was also verified by simplified finite element analysis. Concrete breakout failure occurred only for shallow embedment.
- A minimum leg extension, e_h , of $6d_a$ was estimated as being necessary to develop the tensile strength of the steel, assuming the embedment depth is greater than that which would cause a concrete breakout. The standard leg extension typically supplied with L-bolts is about $4.5 d_a$.
- A minimum embedment depth needed to force a pullout or steel failure depends on e_h , the surface roughness condition of the anchor, and f_c' . However, based on the equations presented in the remaining part of the paper, if the leg extension, e_h , exceeds about $6d_a$, a slip failure can be prevented.
- A statistical significance test was performed to see if anchors embedded in grout and concrete materials belonged to the same population. Standard procedures to evaluate statistical significance of anchor data¹³ were used. The hypothesis evaluated was that the two groups of data are from the same population. At a Confidence Level of 95% and with 53 degrees of freedom, the hypothesis that the data are from the same population is accepted; calculated t-statistic: $t_0 = 1.66$, from statistical table of t-statistics: $c = 1.67$.

Analysis of the data in 1993 resulted in the following average uncracked behavior equations, where the input parameters are consistent with the calculated units.

Concrete breakout strength

$$\begin{aligned} N_c &= 55 (f_c')^{1/2} (h_{ef}) \text{ [lbs]} \\ N_c &= 116 (f_c')^{1/2} (h_{ef}) \text{ [N]} \end{aligned} \quad (\text{Eq. 3})$$

Pullout strength

$$\begin{aligned} N_{\text{slip}} &= 28 (f_c')^{1/2} (e_h - d_a)^2 + 570 (f_i) \pi (h_{\text{ef}} + e_h - d_a) (d_a) \text{ [lbs]} \\ N_{\text{slip}} &= 2.32 (f_c')^{1/2} (e_h - d_a)^2 + 4.02 (f_i) \pi (h_{\text{ef}} + e_h - d_a) (d_a) \text{ [N]} \end{aligned} \quad (\text{Eq. 4})$$

The first term in Equation 4 is a bearing term and the second term is a bond term. The first term should not be taken less than $9600 d_a^2$ [4080 (d_a^2)] nor have e_h greater than $6d_a$.

Steel strength

$$\begin{aligned} N_s &= 0.85 (d_a)^2 (f_{\text{uta}}) \text{ [lbs]} \\ N_s &= 0.85 (d_a)^2 (f_{\text{uta}}) \text{ [N]} \end{aligned} \quad (\text{Eq. 5})$$

where:

f_c' = concrete cylinder compressive strength, psi (MPa)

f_{uta} = ultimate tensile strength of the steel, psi (MPa)

e_h = distance from the inner surface of the L-bolt shaft to the outer tip of the L-bolt [hook extension], inches (mm)

d_a = outside diameter of hooked L-bolt, inches (mm)

h_{ef} = effective embedment depth, distance down from surface of concrete to where the tension load is transferred to the concrete, inches (mm)

f_i = friction index for bond = $1 - \alpha$

α = embedded fraction of anchor that is greased or wrapped to void bond, varies from 0 to 1.0; 0 applies to anchors fully bonded

Geometric definitions of e_h , d_a , and h_{ef} are shown in Figure 4a.

Discussion of WJE test results and analysis—The equation for breakout strength, Equation 3, was calibrated based on the test data plus results of a simplified finite element analysis. The finite element analysis indicated that for anchors with shallow embedment, an elliptical pattern of principal tensile stresses appears at the surface of the concrete, a crack appears parallel to the horizontal leg of the L-shaped anchor bolt, and the concrete breakout failure cone on the surface of the concrete is elliptical in shape. A tensile stress at the concrete surface develops principally transverse to the leg extension direction. The magnitude of the tensile stress was found to correlate with the embedment depth, h_{ef} , instead of h_{ef}^2 and $h_{\text{ef}}^{3/2}$ as is the case for the 45 and 35 degree models, respectively, for breakout of headed anchors.

The equation for pullout/slip has two terms. These terms were developed as follows:

Bearing term: The hook extension is similar to a dowel embedded in concrete and loaded laterally. Research by Vintzeleou⁹ pertaining to the shear behavior of dowels embedded in concrete was used to develop the form of the equation. The Vintzeleou research predicts a dowel capacity of D_u (kips) = $1.3 d_a^2 \sqrt{f_c' f_y}$ [D_u (kN) = $0.55 d_a^2 \sqrt{f_c' f_y}$] where d_a is the dowel diameter (in., mm), f_c' is the concrete compressive strength (ksi, MPa), and f_y is the yield strength of the dowel steel (ksi, MPa) and. Assuming a steel yield strength of 60 ksi (414 MPa), then D_u (kips) = $10 d_a^2 \sqrt{f_c'}$ [D_u (kN) = $10 d_a^2 \sqrt{f_c'}$]. If the dowel extension is too short, then the bearing cannot be developed. In this case, the dowel strength is limited by the plastic moment capacity of the steel, hence the “9600 (d_a^2) [4080 (d_a^2)]” limiting term for the bearing part of Equation 4. The length of the bearing is taken as “ $e_h - d_a$ ” to account for the reduced dowel behavior in the region of the hook bend.

Bond term: The bond strength of smooth bars in tension is usually taken as a maximum bond stress times the contact area. In literature pertaining to bond of smooth steel bars, maximum bond stresses are usually found to be about 250 to 300 psi [1.72 to 2.1 MPa]. Historically, the bond strength of smooth bars in concrete has been taken as a function of the bond area, concrete strength, and a percentage of the concrete strength¹⁴ rather than to assume a constant bond stress.

However, in the case of L-bolts, the leg extension provides a transverse force, causing an increase in the maximum bond stress, similar to the concept of shear friction. Similarly, the tension force on the anchor causes a transverse force to be applied to the extension, increasing its maximum bond stress. Once the anchor starts to pull out, the transverse

force is reduced, but the anchor is no longer a smooth dowel. Instead, it has a bump where the hook bend used to be (see Figure 4d). This bump presses against the side of the hole, causing an apparent increase in the bond stress. The overall length of the anchor in bond is reduced by d_a to account for the bend region. The coefficient “570 [4.02]” was calibrated based on the test data, and, in particular, comparisons between tests on “as-received” bars and bars coated with Teflon tape.

ACI 318 code pullout equation

The current ACI design equation for hooked-bolt anchors makes reference to a relatively obscure report by Kuhn and Shaikh.⁶ The data in Reference 6 is essentially the database assembled by WJE in Reference 10. In Reference 6, the average ultimate strength of a hooked-bolt anchor was proposed as the sum of the bond component and the bearing component, as shown in Equation 6. The coefficients in this equation are based on the Kuhn and Shaikh analysis of test results in the database in Table 1 for hooked anchors. The Kuhn and Shaikh predicted failure loads in uncracked concrete, by Equation 6, are compared to the pullout test results of Table 1 in Figure 6.

$$\begin{aligned} N_{p, \text{average uncracked}} &= 1.5 f_c' e_h d_a + 300 \pi (h_{ef}) (d_a) \quad [\text{lbs}] \\ N_{p, \text{average uncracked}} &= 1.5 f_c' e_h d_a + 2.07 \pi (h_{ef}) (d_a) \quad [\text{N}] \end{aligned} \quad (\text{Eq. 6})$$

where the equation terms have been previously defined.

The design equation in ACI 318-11 Appendix D, Equation D-15,⁵ is a condition representing a hooked-bar anchor in cracked concrete. All design equations in Appendix D are also characteristic load values that are a percentage less than the average value. The amount less than the average depends upon the standard deviation of the test data around the mean. Because the bearing part of Eq. 6 cannot take effect unless the bond strength is exceeded, one approach is to appropriately ignore the second term of Eq. 6. Assuming the second term is ignored can result in a characteristic design equation for the bearing strength of about 75 percent of the average uncracked value, if the coefficient of variation of the data is about 15 percent. A possible characteristic equation for the bearing strength of the hooked-bar anchor is:

$$\begin{aligned} N_{p, 5\% \text{ fractile uncracked}} &= 0.75 (N_{p, \text{average uncracked}}) = 1.25 f_c' e_h d_a \quad [\text{lbs}] \\ N_{p, 5\% \text{ fractile uncracked}} &= 0.75 (N_{p, \text{average uncracked}}) = 1.25 f_c' e_h d_a \quad [\text{N}] \end{aligned} \quad (\text{Eq. 7})$$

Multiplying by the reciprocal of the typical factor to account for concrete cracking, that is, $1/1.4$, results in a close approximation of the ACI 318-11, Appendix D - Equation (D-15):⁵

$$\begin{aligned} N_{p, \text{design cracked}} &= (1/1.4) (1.25 f_c' e_h d_a) \sim 0.9 f_c' e_h d_a \quad [\text{lbs}] \\ N_{p, \text{design cracked}} &= (1/1.4) (1.25 f_c' e_h d_a) \sim 0.9 f_c' e_h d_a \quad [\text{N}] \end{aligned} \quad (\text{Eq. 8})$$

If in Figure 6 the bond portion of the prediction equation is ignored, design based only on bearing is obviously conservative.

Re-evaluation of test results

The remainder of this paper will concentrate on pullout failures and not discuss the other failure modes. Although the ACI 318 design equation is a bearing equation, it is appropriate to ask whether the data on hooked-bolt pullout failures is predominated by bearing, bond, or can be predicted by some combination of both behaviors. Further review of the test results in Table 1 seems appropriate considering the conservative nature of the design equation in ACI 318 as reflected in Figure 6. There are 97 test results listed in Table 1. The reported failures are steel rupture, concrete breakout, pullout, or are unknown. Table 2 summarizes the available results in the database and the range of variables in the entire database and in the pullout failure category. There are 60 pullout failures reported in Table 1. However, five of the pullout failures had teflon wrapped around the shank of the anchor when it was cast into the concrete. The objective of the tests with the teflon wrap was to eliminate the bond and test only the bearing of the hook on the concrete. Review of the load-deflection plots for these five tests showed that the addition of the teflon wrap significantly affected the overall behavior and reduced the ultimate load. These tests were considered biased and eliminated from the pullout test group. The range of the variables in the pullout group appears sufficiently broad to qualify as unbiased toward any specific geometric or material condition. A statistical significance test was performed on the ratio of the test result to calculated result to see if the grout and concrete materials belonged to the

same population. The average grout strength was corrected using the square root of the two strengths to adjust the average grout strength to the average concrete strength. The hypothesis that the two groups of data are in the same population was verified, and therefore, the grout tests are included with the concrete tests in the analysis.

Possible prediction models

The assumption in Eq. 6 is that the pullout strength is the additive contribution of the bond strength of the anchor shank to the concrete and the bearing strength of the hook. This approach has been taken in the past for hooked reinforcing bars but was abandoned in the early 1980s. Again in Figure 6, the comparison of the Eq. 6 prediction to the test data shows that the Eq. 6 is conservative and the 5% fractile is about 2/3 the average value predicted by Eq. 6.

Two plots were made to examine whether the pullout strength is dominated by bond or bearing. In Figures 7 and 8, the test failure loads are plotted against the bond area or a pseudo bearing area, respectively. Review of these two graphs does not indicate that bond or bearing area, when considered separately, offers a distinctly better correlation of the measured test results to one set of variables over the other, although the bearing data does almost statistically go through a zero origin.

Several different models were subsequently investigated. Prediction models of the format shown below in Eq. 9 were considered.

$$N_p = C (d_a)^\alpha (e_h)^\beta (h_{ef})^\gamma (f_c')^\delta \quad (\text{Eq. 9})$$

where:

C = linear correlation coefficient

α = exponent on the shank diameter (d_a) of the anchor

β = exponent on the hook leg extension (e_h)

γ = exponent on the effective embedment depth (h_{ef}) of the anchor

δ = exponent on the compressive strength (f_c') of concrete

Multiple linear regression analyses, least squares with multiple independent variables, were made using the format in Eq. 9. Results are listed in Table 3 as Trials 1, 2, and 3 for bearing, bond, and bond/bearing models, respectively.

An alternate approach is to assume an additive model as done by Kuhn and Shaikh and in Reference 6. The format in Eq. 10 was used for a least squares fit.

$$N_p = C_1 + C_2 (f_c' d_a e_h) + C_3 (f_c' \pi d_a h_{ef}) \quad (\text{Eq. 10})$$

where:

C_1 = linear regression intercept

C_2 = correlation coefficient on bearing stress component

C_3 = correlation coefficient on bond stress component

Regression coefficients for the Eq. 10 format are given as Trial 4 in Table 3.

Discussion of least squares prediction models

The authors' review of the statistics associated with the different models investigated indicates that a bearing model, Trial 1, is a promising model because the exponents on the diameter and the hook leg extension are very near 1, as one would expect if load is controlled by a "bearing area." The exponent on the concrete strength, however, is very small implying that the compressive strength of the concrete is not an influential variable in predicting the pullout failure load for hooked-bar anchors in this database. The regression equation using terms that should represent the bond area, Trial 2, resulted in exponents that have no relation to the bond area. The exponent of the compressive strength is also 0.3, again implying compressive strength is not a very influential variable. This does not mean the Trial 2 equation is inappropriate to predict pullout capacity, it only says that the terms we normally expect to represent bond have exponents other than one. Equations representing the combination of bond and bearing, either in the exponential or addition form are, likewise, reasonable predictors of strength. All of the regression equations had correlation coefficients (R) that are near 0.9 or above. For anchor strength prediction equations, R -values of 0.9 are considered good.

SIMPLIFIED DESIGN MODELS

Regression equations have coefficients that numerically give the best fit to the data. However, for design, simpler equations and rounded coefficients are desirable. Additionally, the characteristic design equation is taken as a percentage, 5 percent fractile, of the equation representing the average strength. Each of the trial equations were simplified, exponents rounded, and the 5 percent fractile value determined. One of the trial exponential equations was found to have very small value for the exponent on the compressive strength of the concrete and therefore the contribution of the concrete strength was ignored. The addition equation representing contributions for bearing and bond also had a regression intercept which was ignored for simplification. Results of simplifying the design models are listed below and the statistics associated with the simplification are summarized in Table 4.

$$\begin{aligned} N_{p, \text{average bearing uncracked}} &= 9500 d_a e_h \quad [\text{lbs}] \\ N_{p, \text{average bearing uncracked}} &= 65.5 d_a e_h \quad [\text{N}] \end{aligned} \quad (\text{Eq. 11})$$

$$\begin{aligned} N_{p, \text{average bond uncracked}} &= 1600 d_a^2 h_{ef}^{1/2} f_c'^{1/4} \quad [\text{lbs}] \\ N_{p, \text{average bond uncracked}} &= 7.6 d_a^2 h_{ef}^{1/2} f_c'^{1/4} \quad [\text{N}] \end{aligned} \quad (\text{Eq. 12})$$

$$\begin{aligned} N_{p, \text{average bond/bearing uncracked}} &= 1200 d_a^{3/4} e_h h_{ef}^{1/2} f_c'^{1/8} \quad [\text{lbs}] \\ N_{p, \text{average bond/bearing uncracked}} &= 4.33 d_a^{3/4} e_h h_{ef}^{1/2} f_c'^{1/8} \quad [\text{N}] \end{aligned} \quad (\text{Eq. 13})$$

$$\begin{aligned} N_{p, \text{average (bond + bearing) uncracked}} &= 2.20 f_c' d_a e_h + 0.08 f_c' \pi d_a h_{ef} \quad [\text{lbs}] \\ N_{p, \text{average (bond + bearing) uncracked}} &= 2.20 f_c' d_a e_h + 0.08 f_c' \pi d_a h_{ef} \quad [\text{N}] \end{aligned} \quad (\text{Eq. 14})$$

where the variables are f_c' (psi) [MPa], d_a (in.) [mm], e_h (in.) [mm], and h_{ef} (in.) [mm].

Plots of the test to predicted ratio for each of these models are shown in Figure 9. All are reasonable predictors of the design strength. Equation 11 has the largest 5 percent fractile value and Equation 14 has the lowest.

The authors' believe the prediction models listed above in Equations 11 - 14 are better predictors of the pullout strength of L-bolts than what is in the ACI 318-11 code. The question is, which equation most appropriately represents a model of the behavior of the hooked-bolt anchor. It is appropriate, as apparently done to arrive at the current ACI 318 design pullout strength of hooked-bar anchors, not to use Eq. 14 because the bearing cannot exist unless the bond is broken. Equation 12, although an equation that has variables that influence bond, does not have exponents that are statistically close to 1 and the contribution of the concrete strength is small because of the 1/4 exponent. The bond/bearing model, Eq. 13, has one design advantage in that it permits a determination of how deep to embed the anchor. According to Figure 9c, the average of the predicted strengths is very near 1.0. This trend is very similar to that shown in Figure 9a, which is a plot of Eq. 11. Equation 11 is, however, very simple.

The compressive strength of the embedding material ranges from 2000 psi to 5000 psi. The analysis of the data strongly indicates that the compressive strength of the embedding material is not a significant variable. Therefore, extrapolation to compressive strengths greater than the range tested should be conservative.

A comparison of the measured uncracked test load to the predicted uncracked test load by Eq. 11 and Eq. 13 are shown plotted in Figure 10 and Figure 11, respectively. The existing uncracked ACI code strength prediction, that is, Eq. 8 without the bond portion, is about half the actual measured strength. Therefore, an increase in the pullout capacity for L-bolts seems warranted. It is recommended that a design equation of the form in Eq. 15 or Eq. 16 be adopted. In addition, it is recommended that the leg extension requirements in the ACI 318 code remain and that the minimum embedment depth for all hooked-bar anchors be $15d_a$ if the design is by Eq. 15. Since embedment depth is a variable in Eq. 16, the same restriction does not apply. In no case should the value of the predicted strength be taken larger than the steel strength. Both the concrete breakout strength and the steel strength should also be compared to the pullout strength to see which failure mode controls the strength.

$$\begin{aligned} N_{p, 5\% \text{ fractile design uncracked}} &= 5400 d_a e_h \quad [\text{lbs}] \\ N_{p, 5\% \text{ fractile design uncracked}} &= 37 d_a e_h \quad [\text{N}] \end{aligned} \quad (\text{Eq. 15})$$

$$N_{p, 5\% \text{ fractile design uncracked}} = 750 d_a^{3/4} e_h h_{ef}^{1/2} f_c'^{1/8} \quad [\text{lbs}] \quad (\text{Eq. 16})$$

$$N_{p, 5\% \text{ fractile design uncracked}} = 2.7 d_a^{3/4} e_h h_{ef}^{1/2} f_c^{1/8} \quad [N]$$

where the variables are f_c' (psi)[MPa], d_a (in.)(mm), e_h (in.)(mm), and h_{ef} (in.)(mm).

SUMMARY

Testing data has existed on the pullout strength of L-bolt anchors for many years but the data has not been thoroughly analyzed and published. Results reported in this paper indicate that the pullout failure design strength for L-bolt anchors could be increased and still be consistent with the provisions in the ACI code that allow design models to be in substantial agreement with comprehensive tests. The authors' believe the results presented herein are comprehensive and meet the requirements of the code provisions. Equations representing the pullout strength of L-bolt anchors correlate well with the test data. The existing pullout failure load model for a hooked anchor is very conservative and can be increased by almost a factor of two.

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TABLES

Table 1 - Database of hooked-bar anchors

No.	ID	Diameter d_a (in)	Embedment depth h_{ef} (in)	Leg extension e_h (in)	Embedment depth ratio h_{ef} / d_a	Leg extension ratio e_h / d_a	Threads per inch (n)	f_c' (psi)	f_{su} (f_y) (ksi)	P_{Test} (kips)	Observed failure mode
WJE Data											
1	2-1	0.688	7.500	3.125	10.9	4.54	11	1980	87.5	18.91	pullout
2	2-2	0.688	9.500	3.125	13.8	4.54	11	1980	87.5	21.61	pullout
3	2-3	0.688	10.800	3.125	15.7	4.54	11	1980	87.5	21.69	pullout
4	2-4	0.688	6.000	3.125	8.7	4.54	11	1980	87.5	12.50	breakout
5	2-5	0.688	4.750	3.125	6.9	4.54	11	1980	87.5	10.68	breakout
6	3-1	1.000	6.000	4.500	6.0	4.50	8	3730	84.5	19.63	breakout
7	3-2	1.000	8.750	4.500	8.8	4.50	8	3730	84.5	25.70	breakout
8	3-3	1.000	12.000	4.500	12.0	4.50	8	3730	84.5	47.28	pullout-yielding
9	3-4	1.000	12.000	4.500	12.0	4.50	8	3730	84.5	18.21	pullout-teflon
10	3-5	0.500	3.000	2.250	6.0	4.50	13	3730	78.3	8.99	pullout
11	3-6	0.500	4.500	2.250	9.0	4.50	13	3730	76.1	10.94	steel
12	3-7	0.500	6.000	2.250	12.0	4.50	13	3730	76.1	10.35	steel
13	3-8	0.500	6.000	2.250	12.0	4.50	13	3730	76.1	4.87	pullout
14	3-9	0.500	9.000	2.250	18.0	4.50	13	3730	76.1	11.33	steel
15	3-10	0.500	12.000	2.250	24.0	4.50	13	3730	76.1	11.06	steel
16	3-11	0.680	9.000	1.750	13.2	2.57	11	3730	87.1	15.87	pullout
17	3-12	0.680	13.500	1.750	19.9	2.57	11	3730	87.1	17.20	pullout
18	3-13	0.680	9.000	5.875	13.2	8.64	11	3730	87.1	28.00	steel
19	3-14	0.680	13.500	5.875	19.9	8.64	11	3730	87.1	29.74	steel
20	3-15	0.680	6.750	3.125	9.9	4.60	11	3730	87.1	21.82	breakout
21	3-16	0.680	6.750	3.125	9.9	4.60	11	3730	87.1	11.63	pullout-teflon
22	3-17	0.680	9.000	3.125	13.2	4.60	11	3730	87.1	28.54	steel
23	3-18	0.680	9.000	3.125	13.2	4.60	11	3730	87.1	10.54	pullout-teflon
24	3-19	0.680	13.500	3.125	19.9	4.60	11	3730	87.1	28.01	steel
25	3-20	0.680	13.500	3.125	19.9	4.60	11	3730	87.1	11.59	pullout-teflon
26	4-1	0.685	2.750	3.125	4.0	4.56	11	5460	89.1	12.22	breakout
27	4-2	0.683	5.500	3.125	8.1	4.58	11	5460	89.1	24.43	breakout
28	4-3	0.682	8.250	3.125	12.1	4.58	11	5460	89.1	29.09	steel
29	4-4	0.680	11.000	3.125	16.2	4.60	11	5460	87.5	29.30	steel
30	4-5	0.688	13.750	3.125	20.0	4.54	11	5460	89.1	15.51	nut strip
31	4-6	0.687	2.750	3.125	4.0	4.55	11	5460	89.1	11.73	breakout
32	4-7	0.675	4.125	3.125	6.1	4.63	11	5460	87.1	17.97	breakout
33	4-8	0.675	5.500	3.125	8.1	4.63	11	5460	89.1	25.24	pullout
34	4-9	0.678	8.250	3.125	12.2	4.61	11	5460	89.1	22.86	nut strip
35	4-10	0.680	11.000	3.125	16.2	4.60	11	5460	89.1	29.05	pullout
Nelson Data											
36	1/2Ba	0.500	6.000	2.000	12.0	4.00	13	2950	104.9	10.30	steel
37	1/2Bb	0.500	6.000	2.000	12.0	4.00	13	2950	104.9	10.00	pullout
38	5/8Ba	0.625	8.000	2.000	12.8	3.20	11	2950	97.3	16.30	pullout
39	5/8Bb	0.625	8.000	2.000	12.8	3.20	11	2950	97.3	16.90	pullout
40	3/4Ba	0.750	8.000	3.000	10.7	4.00	10	2950	95.8	21.20	pullout
41	3/4Bb	0.750	8.000	3.000	10.7	4.00	10	2950	95.8	25.00	pullout
42	1Ba	1.000	12.000	4.000	12.0	4.00	8	2950	76.2	36.20	pullout
43	1Bb	1.000	12.000	4.000	12.0	4.00	8	2950	76.2	46.20	steel
University of Texas - low side cover											
44	UT-1	1.250	12.500	2.000	10.0	1.60	7	5160	75.0	47.00	?
45	UT-2	1.250	12.500	2.000	10.0	1.60	7	5160	75.0	47.00	?
46	UT-3	2.000	20.000	4.000	10.0	2.00	5	4710	98.0	82.00	?
47	UT-4	2.000	20.000	4.000	10.0	2.00	5	4710	98.0	128.00	?

Clemson University Data - Table 8.3											
48	1	0.375	6.375	1.125	17.0	3.00	16	2513	92.0	3.58	pullout
49	2	0.375	6.375	1.125	17.0	3.00	16	2513	92.0	4.30	pullout
50	3	0.375	6.375	1.125	17.0	3.00	16	2513	92.0	3.24	pullout
51	4	0.375	6.375	1.125	17.0	3.00	16	2513	92.0	3.02	pullout
52	5	0.375	6.375	1.125	17.0	3.00	16	2513	92.0	3.80	pullout
53	6	0.375	6.375	1.125	17.0	3.00	16	2513	92.0	4.00	pullout
54	7	0.375	6.375	1.125	17.0	3.00	16	2513	92.0	3.60	pullout
55	8	0.500	6.375	1.500	12.8	3.00	13	2513	65.0	9.62	pullout
56	9	0.500	6.375	1.500	12.8	3.00	13	2513	65.0	8.74	pullout
57	10	0.500	6.375	1.500	12.8	3.00	13	2513	65.0	6.37	pullout
58	11	0.500	6.375	1.500	12.8	3.00	13	2513	65.0	8.80	pullout
59	12	0.500	6.375	1.500	12.8	3.00	13	2513	65.0	7.40	pullout
60	13	0.750	6.375	2.250	8.5	3.00	10	2513	67.0	19.38	pullout
61	14	0.750	6.375	2.250	8.5	3.00	10	2513	67.0	16.65	pullout
62	15	0.750	6.375	2.250	8.5	3.00	10	2513	67.0	15.54	pullout
63	16	0.750	6.375	2.250	8.5	3.00	10	2513	67.0	17.20	pullout
64	17	0.750	6.375	2.250	8.5	3.00	10	2513	67.0	19.00	pullout
65	18	0.750	6.375	2.250	8.5	3.00	10	2513	67.0	17.20	pullout
Clemson University Data - Table 8.4											
66	1	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	8.13	pullout
67	2	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	5.88	pullout
68	3	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	6.50	pullout
69	4	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	5.50	pullout
70	5	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	5.26	pullout
71	6	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	7.75	pullout
Clemson University Data - Table 8.5											
72	1	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	5.75	pullout
73	2	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	7.00	pullout
74	3	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	5.00	pullout
75	4	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	5.50	pullout
76	5	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	9.00	pullout
77	6	0.500	5.625	1.500	11.3	3.00	13	3202	65.0	8.80	pullout
78	7	0.750	5.625	2.250	7.5	3.00	10	3202	67.0	12.50	pullout
79	8	0.750	5.625	2.250	7.5	3.00	10	3202	67.0	10.00	pullout
80	9	0.750	5.625	2.250	7.5	3.00	10	3202	67.0	11.75	pullout
81	10	0.750	5.625	2.250	7.5	3.00	10	3202	67.0	9.50	pullout
82	11	0.750	5.625	2.250	7.5	3.00	10	3202	67.0	12.00	pullout
83	12	0.750	5.625	2.250	7.5	3.00	10	3202	67.0	11.00	pullout
Clemson University Data - ASCE paper Table 5											
84	1	1.000	6.375	4.000	6.4	4.00	8	2500	76.0	11.35	breakout
85	2	1.000	6.375	4.000	6.4	4.00	8	2500	76.0	17.58	breakout
86	3	1.000	6.375	4.000	6.4	4.00	8	2500	76.0	18.43	breakout
87	4	1.250	6.375	5.000	5.1	4.00	7	2500	70.0	14.90	breakout
88	5	1.250	6.375	5.000	5.1	4.00	7	2500	70.0	16.05	breakout
89	6	1.250	6.375	5.000	5.1	4.00	7	2500	70.0	14.51	breakout
University of Wisconsin - Milwaukee											
90	1	0.563	2.000	2.500	3.6	4.44	12	6560	69.0	5.00	breakout
91	2	0.563	2.000	2.500	3.6	4.44	12	6560	69.0	8.30	breakout
92	3	0.563	4.000	2.500	7.1	4.44	12	6560	89.7	16.80	steel
93	4	0.563	4.000	2.500	7.1	4.44	12	6560	89.7	17.20	steel
94	5	0.563	6.000	2.500	10.7	4.44	12	3290	69.0	14.60	pullout
95	6	0.563	6.000	2.500	10.7	4.44	12	3290	69.0	15.20	pullout
96	7	0.563	8.000	2.500	14.2	4.44	12	3290	69.0	13.40	pullout
97	8	0.563	8.000	2.500	14.2	4.44	12	3290	69.0	15.00	pullout

Table 2 - Summary of results in hooked-bolt database

Failure type	Category				
	Total steel failures / stripping of nut	Concrete breakout failures	Total pullout failures / Pullout failures with shank wrapped with teflon	Unknown - possibly breakout with small side edge distance	Total
Number of tests	16 / 2	17	60 / 5	4	97

Test variables	Range in Table 1 database	Range for pullout failures
Compressive strength (f'_c), psi (MPa)	1980 - 6560 (13.6 - 45.2)	2,000 - 5,000 (13.8 - 34.5)
Anchor diameter (d_a), in. (mm)	3/8 - 2 (9.5 - 51)	3/8 - 1 (9.5 - 25)
Embedment depth (h_{ef}), in. (mm)	2 - 20 (51 - 508)	3 - 13 1/2 (76 - 343)
Hook extension (e_h), in. (mm)	1 1/8 - 5 7/8 (28.5 - 149)	1 1/8 - 4 1/2 (28.5 - 114)
Embedment depth ratio (h_{ef} / d_a)	3.5 - 24	6.0 - 19.8
Hook extension ratio (e_h / d_a)	1.6 - 8.6	2.5 - 4.6

Table 3 - Summary of least-squares regression analysis on pullout failures of hooked-bolt anchors

Trial No.	Equation model	Units of equation	Number of data points	Regression Coefficient (C)	Exponent on d_a * (α)	Exponent on e_h * (β)	Exponent on h_{ef} * (γ)	Exponent on f'_c ** (δ)	Correlation coefficient (R)
1	Exponential bearing	kips	55	4.573	0.828	1.200	-	0.063	0.941
2	Exponential bond	kips	55	0.884	1.954	-	0.640	0.294	0.916
3	Exponential bond/bearing	kips	55	0.991	0.780	1.070	0.475	0.149	0.935
				Intercept (C_1)	Coefficient of bearing component (C_2)	Coefficient of bond component (C_3)			
4	Addition of bond and bearing components	kips	55	1.511	2.004	0.079			0.912

* d_a , e_h , h_{ef} (in inch dimensions)

** f'_c (in pounds per square inch - psi)

Table 4 - Simplifications to regression equations to predict pullout strength, N_p

Trial No.	Equation model type	N_p , Average equation form (kips)	Ratio of average test to predicted	Standard deviation	Coefficient of variation (percent)	5% fractile strength
1	Exponential bearing	$N_{p,bearing} = 9.5 d_a e_h$	1.008	0.220	21.8	$N_{p,bearing} 5\% = 0.576 N_{p,bearing}$
2	Exponential bond	$N_{p,bond} = 1.6 d_a^2 h_{ef}^{1/2} f'_c^{1/4}$	1.067	0.277	25.9	$N_{p,bond} 5\% = 0.523 N_{p,bond}$
3	Exponential bond/bearing	$N_{p,bond/bearing} = 1.2 d_a^{3/4} e_h h_{ef}^{1/2} f'_c^{1/8}$	1.002	0.193	19.2	$N_{p,bond/bearing} 5\% = 0.624 N_{p,bond/bearing}$
4	Addition of bond and bearing	$N_{p,bond+bearing} = 2.2 f'_c d_a e_h + 0.08 f'_c \pi d_a h_{ef}$	1.071	0.291	27.1	$N_{p,bond+bearing} 5\% = 0.500 N_{p,bond+bearing}$

FIGURES

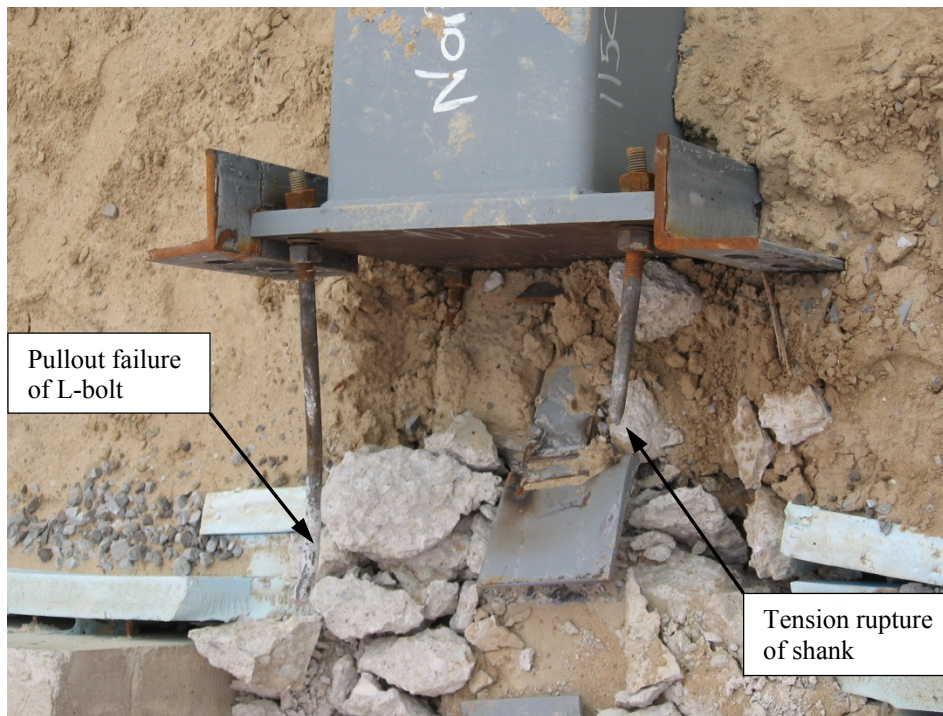


Figure 1 - Tension failures of an L-bolt anchor.

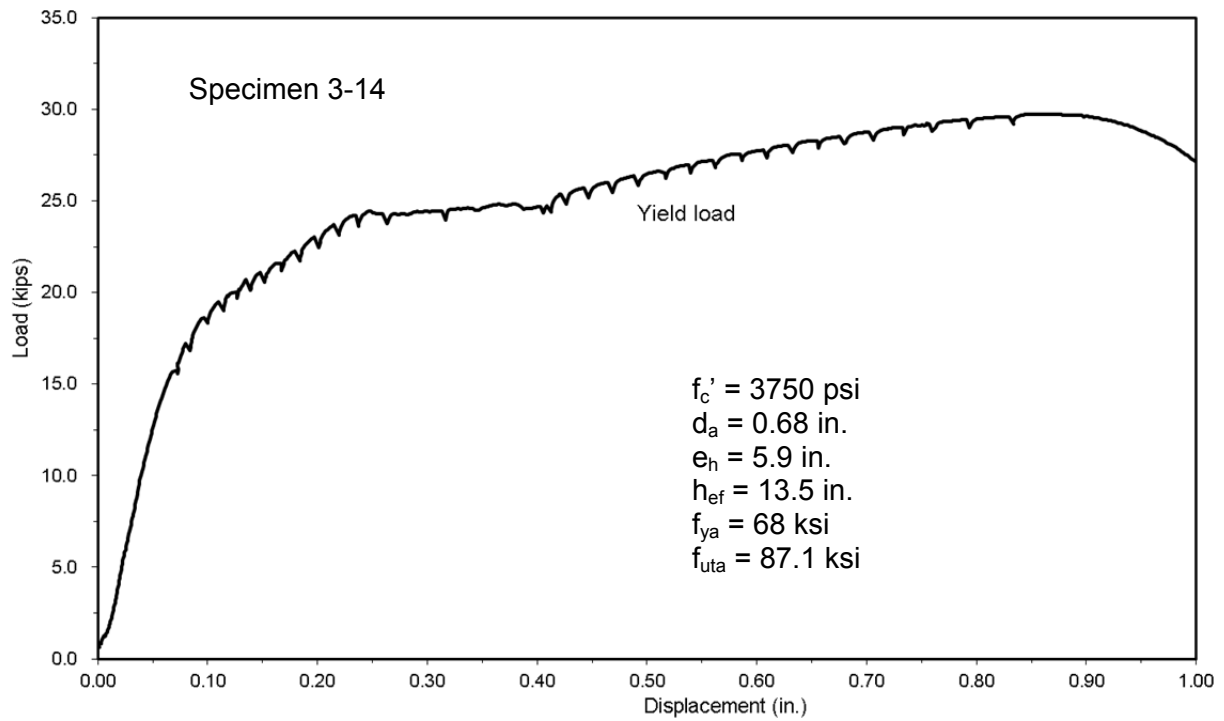


Figure 2a - Load-displacement curve for a steel failure of an L-bolt anchor.

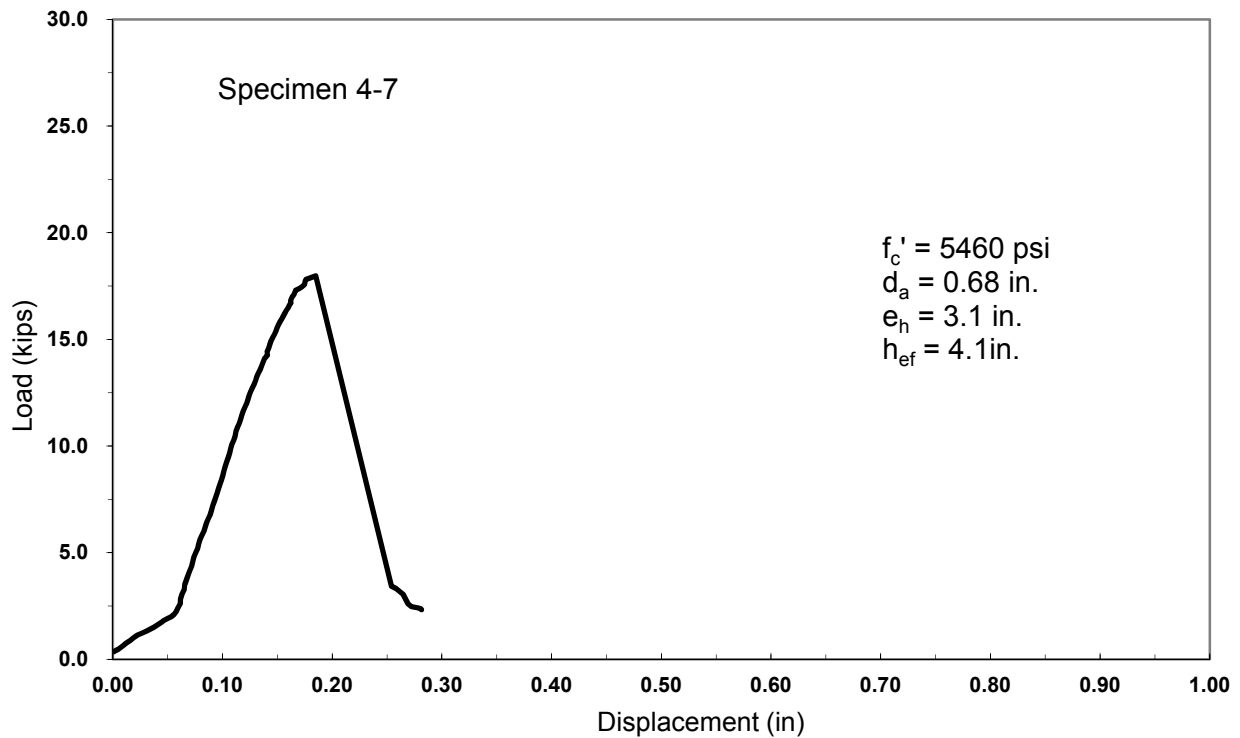


Figure 2b - Load-displacement curve for a concrete breakout failure of an L-bolt anchor.

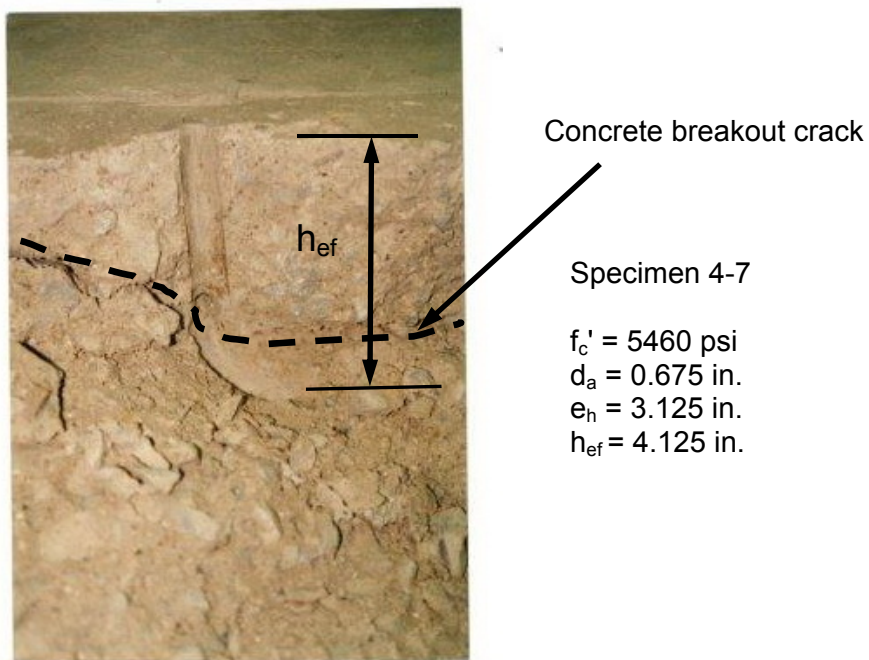


Figure 3 - Tension concrete breakout failure.

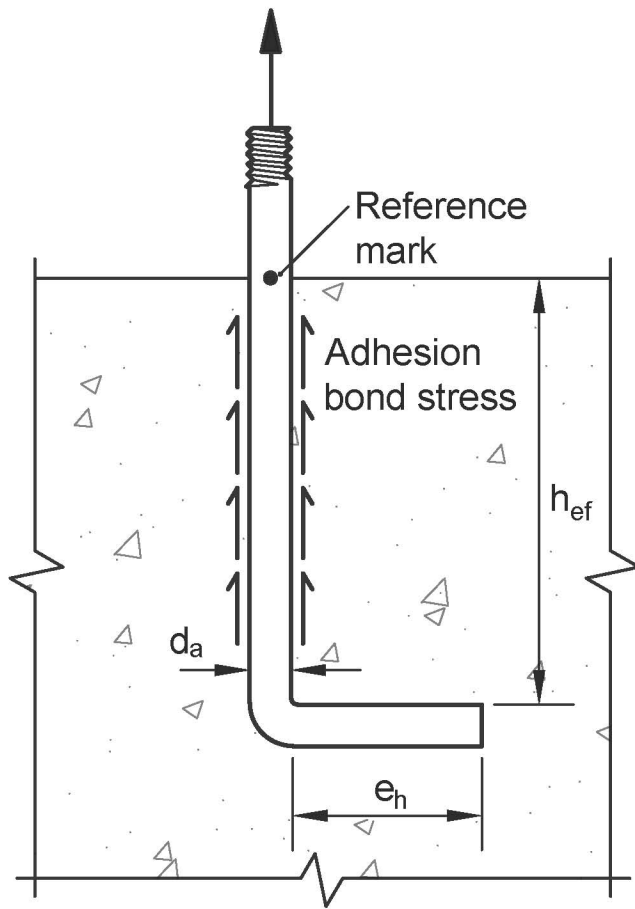


Figure 4a - Stages of a pullout failure: Anchor load transferred by adhesion (bond) to the concrete.

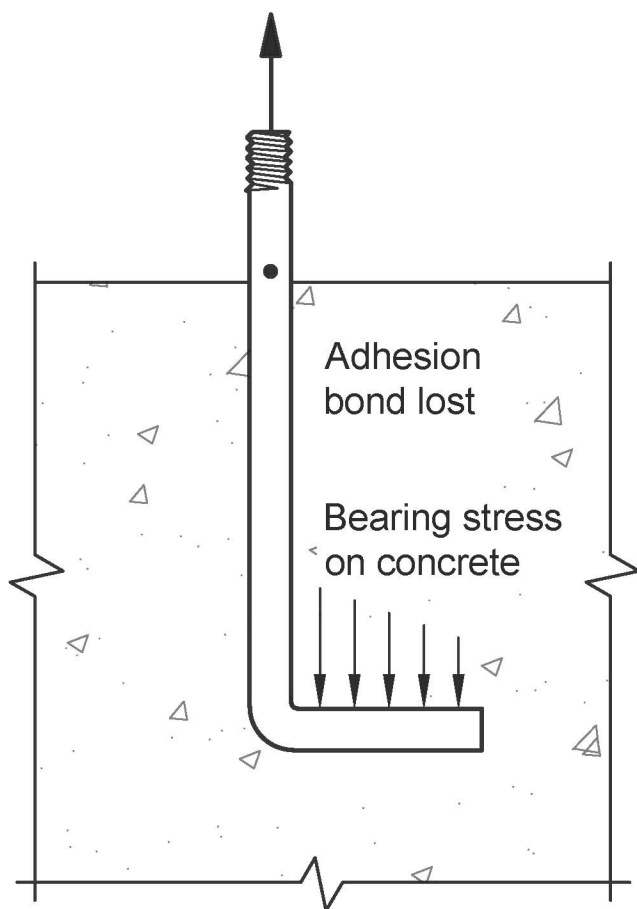


Figure 4b - Stages of a pullout failure: Adhesion breaks and load transfers to bearing on the hook.

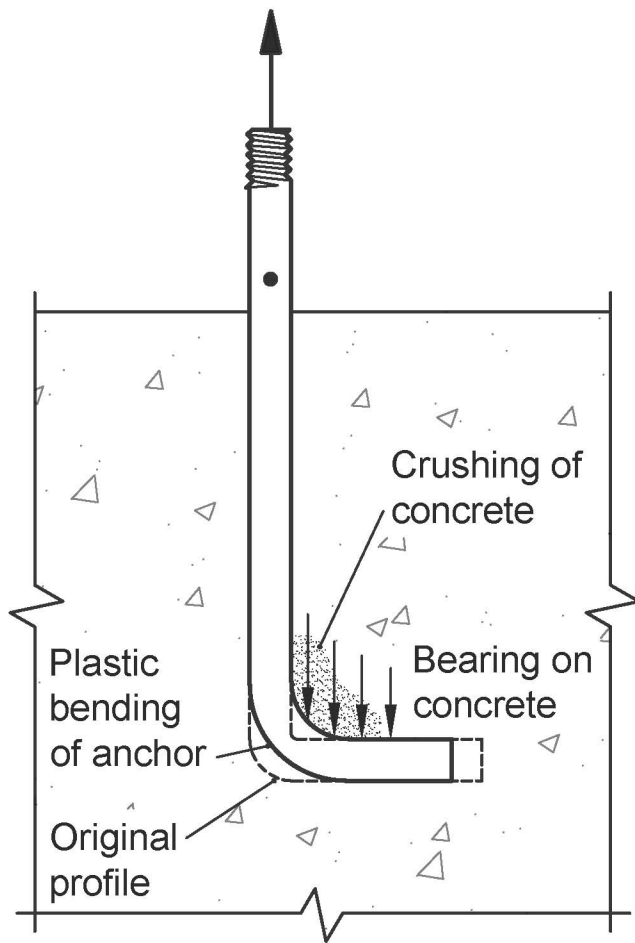


Figure 4c - Stages of a pullout failure: Concrete crushing starts at the hook bend.

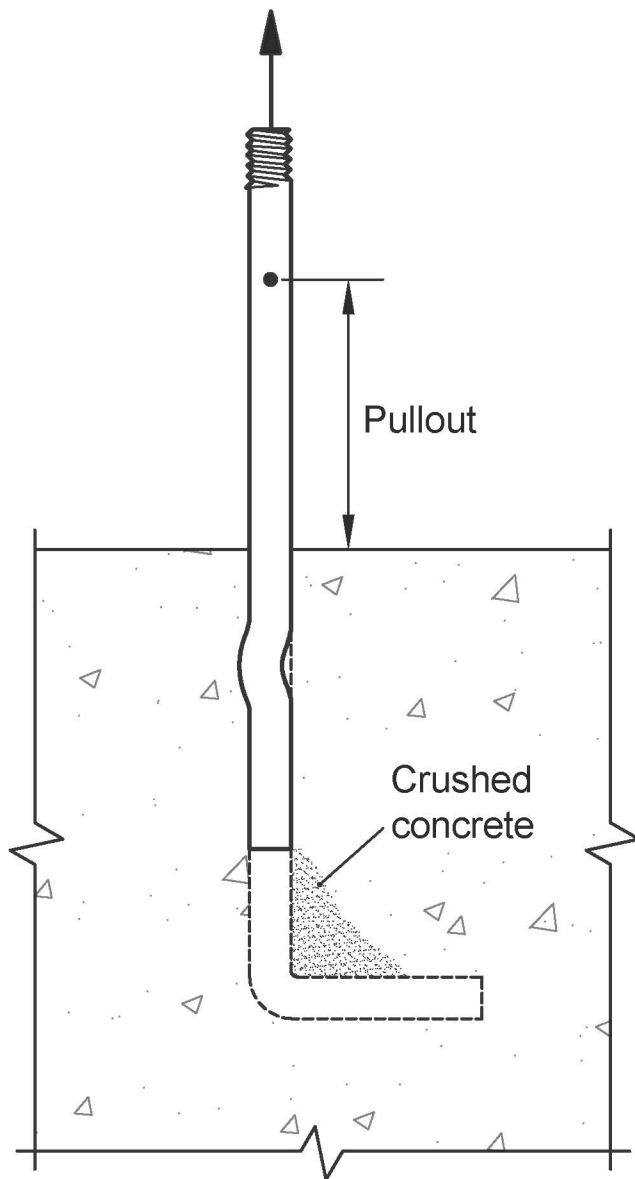


Figure 4d - Stages of a pullout failure: Sufficient crushing of the concrete at the hook bend allows the hook to straighten out and the anchor shank pulls out of the concrete.

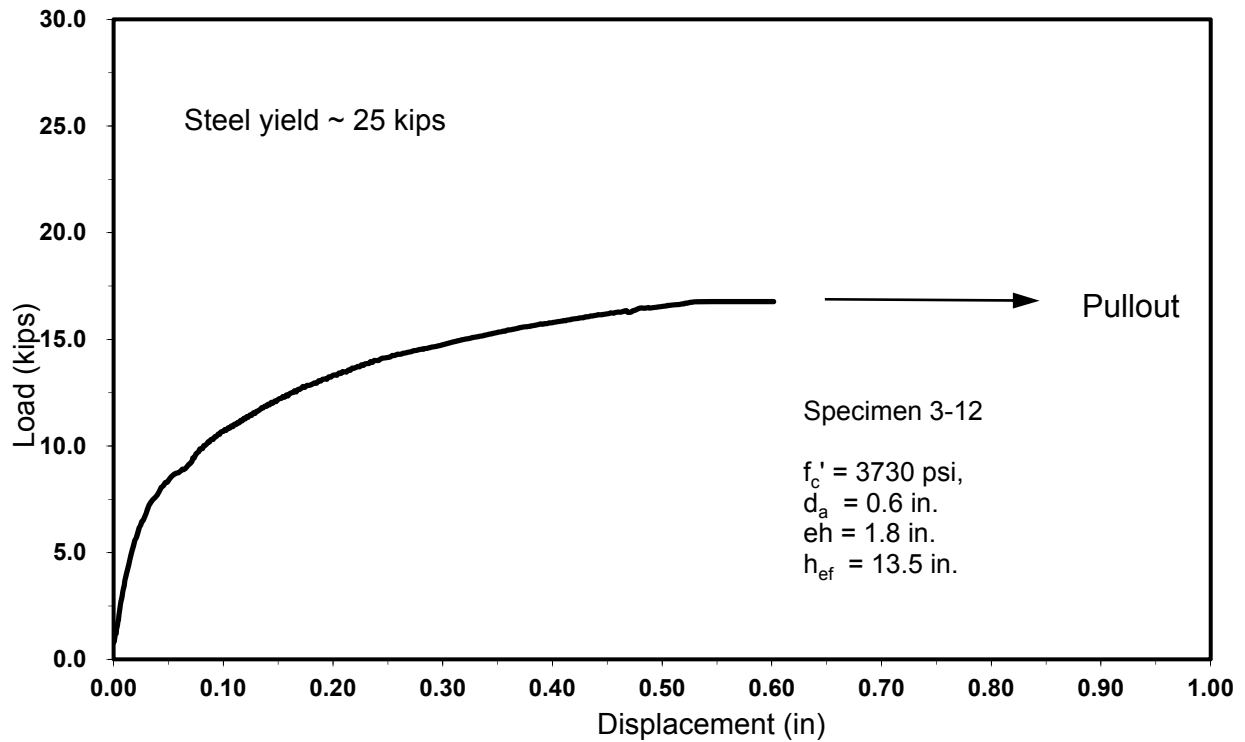


Figure 5 - Load-displacement curve for a pullout failure of an L-bolt anchor.

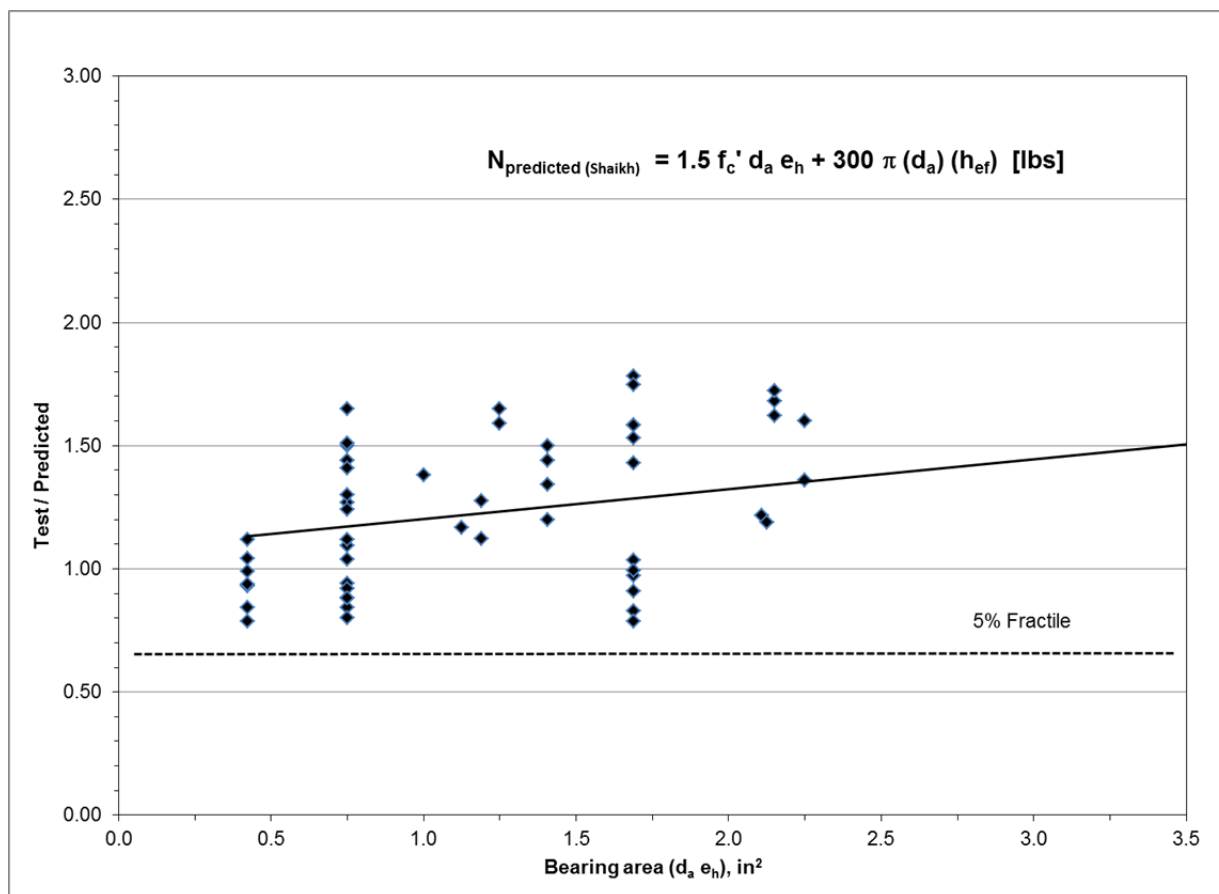


Figure 6 - Comparison of Equation 3 prediction to the pullout database.

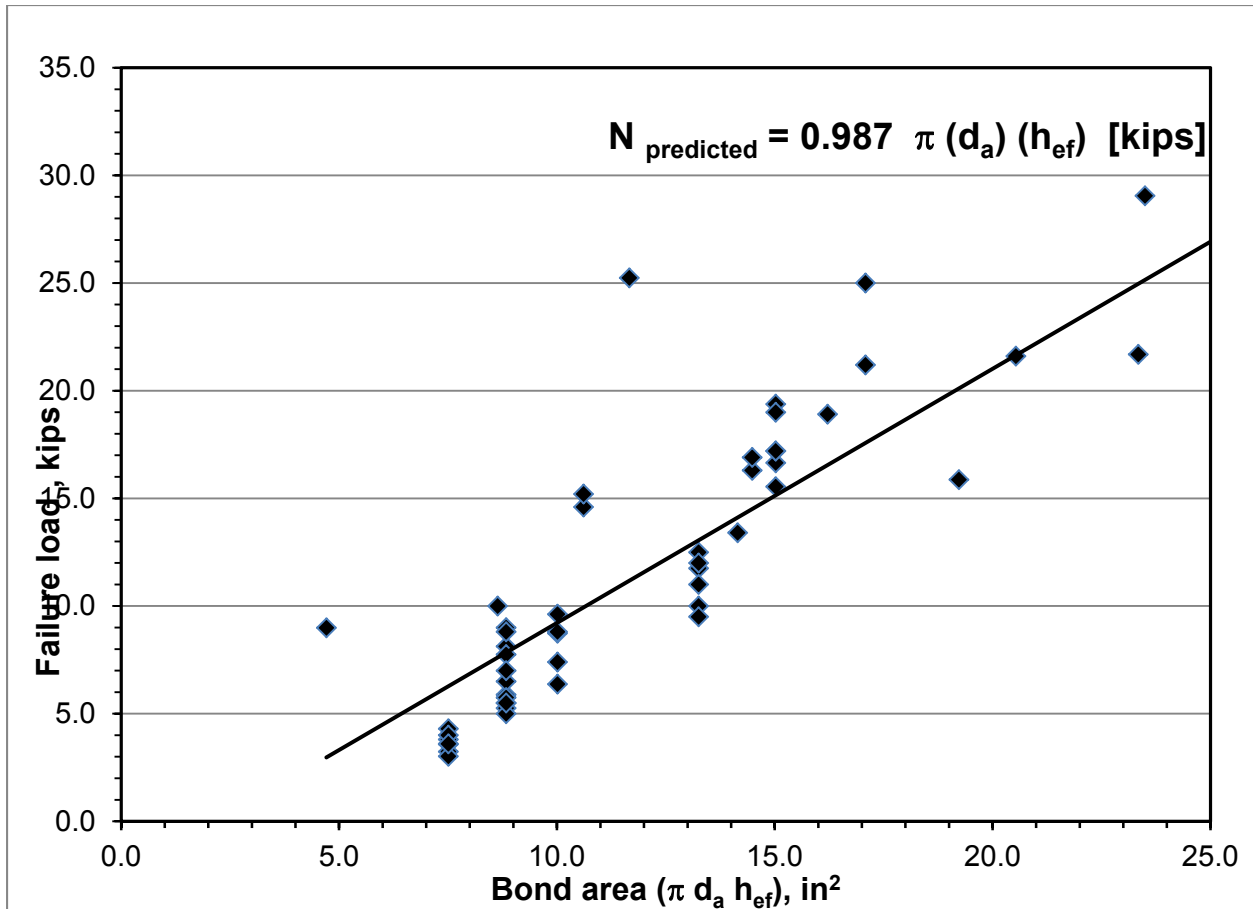


Figure 7 - Pullout test failure load plotted against the bond area of the straight shank of the L-bolt anchor.

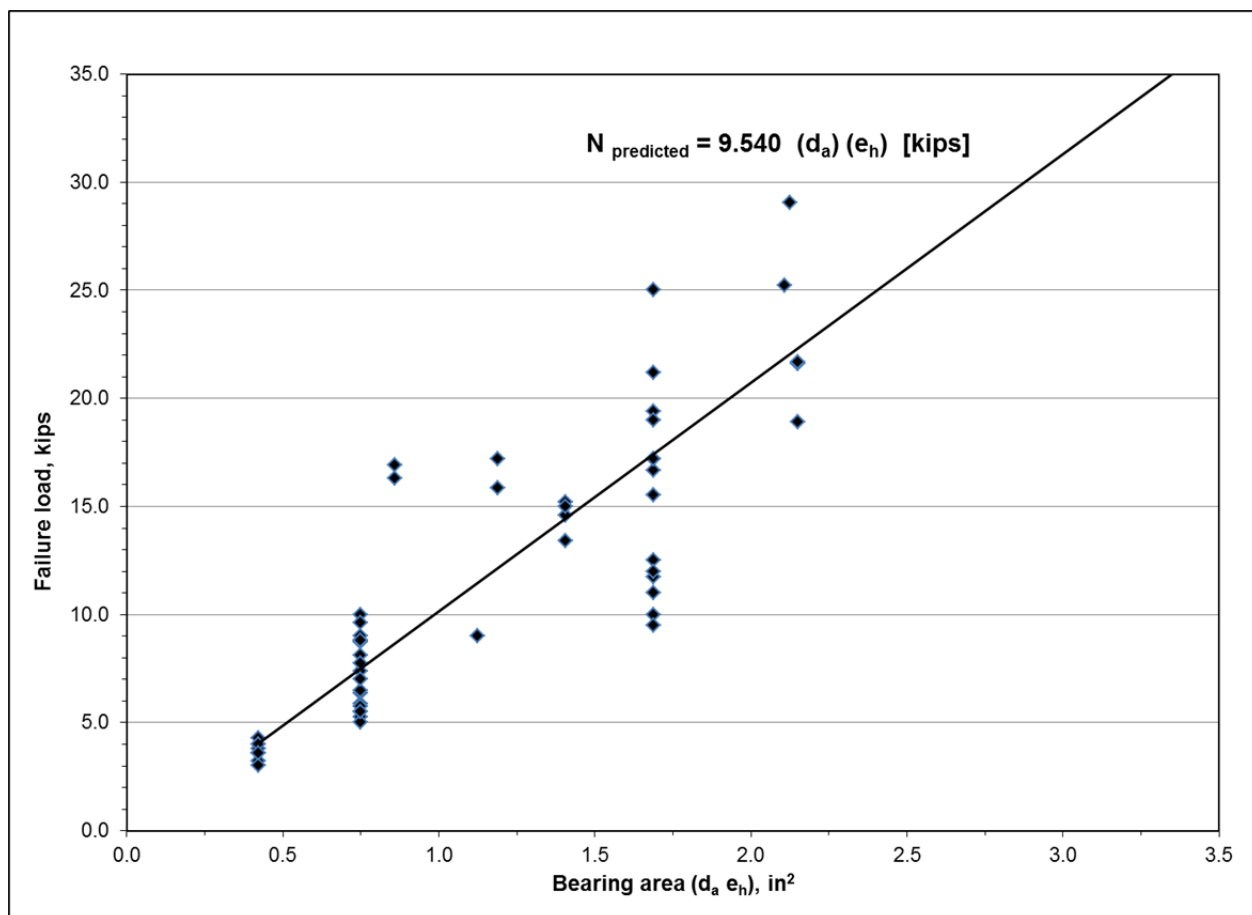


Figure 8 - Pullout test failure load plotted against the bearing area of the hook.

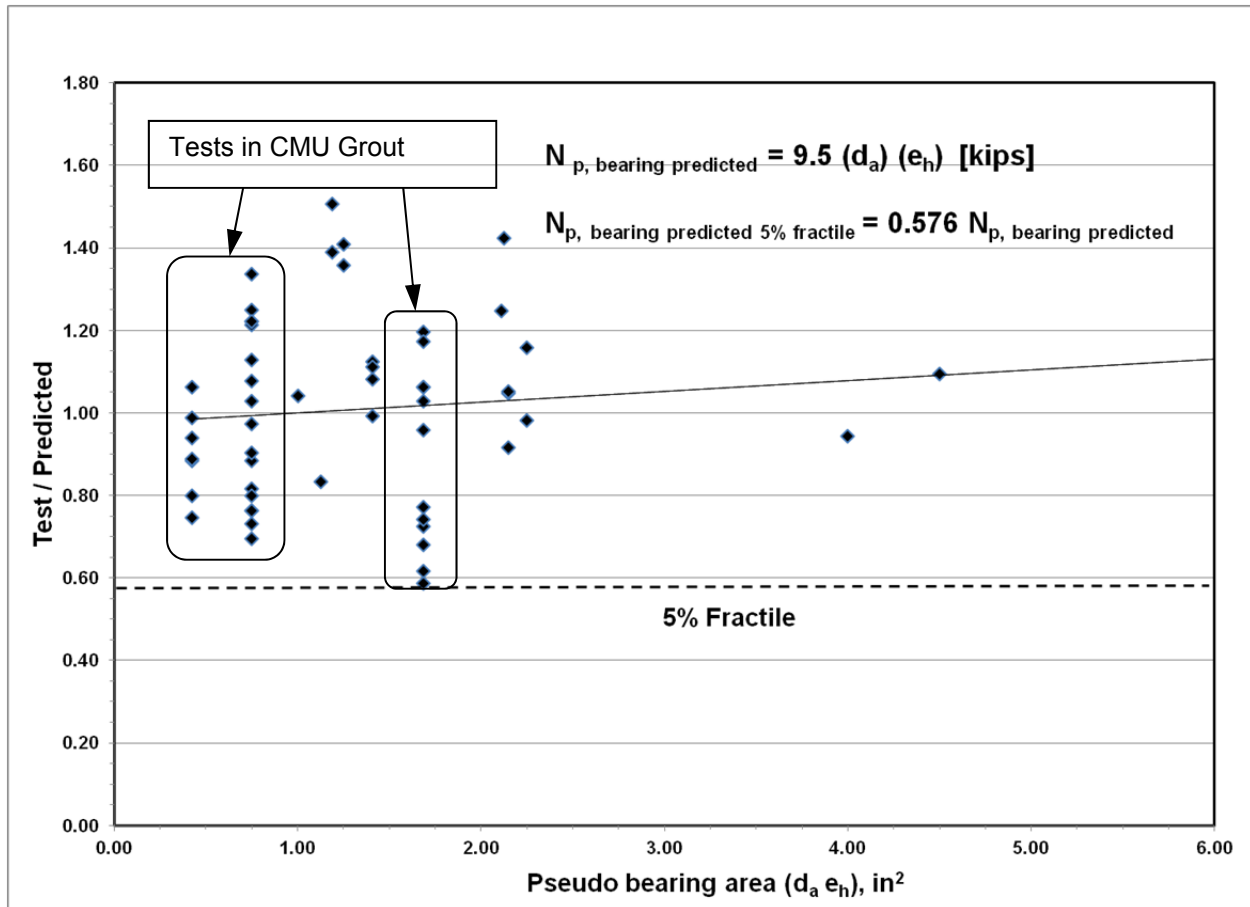


Figure 9a - Simplified average and 5 percent fractile design equation - bearing exponential model.

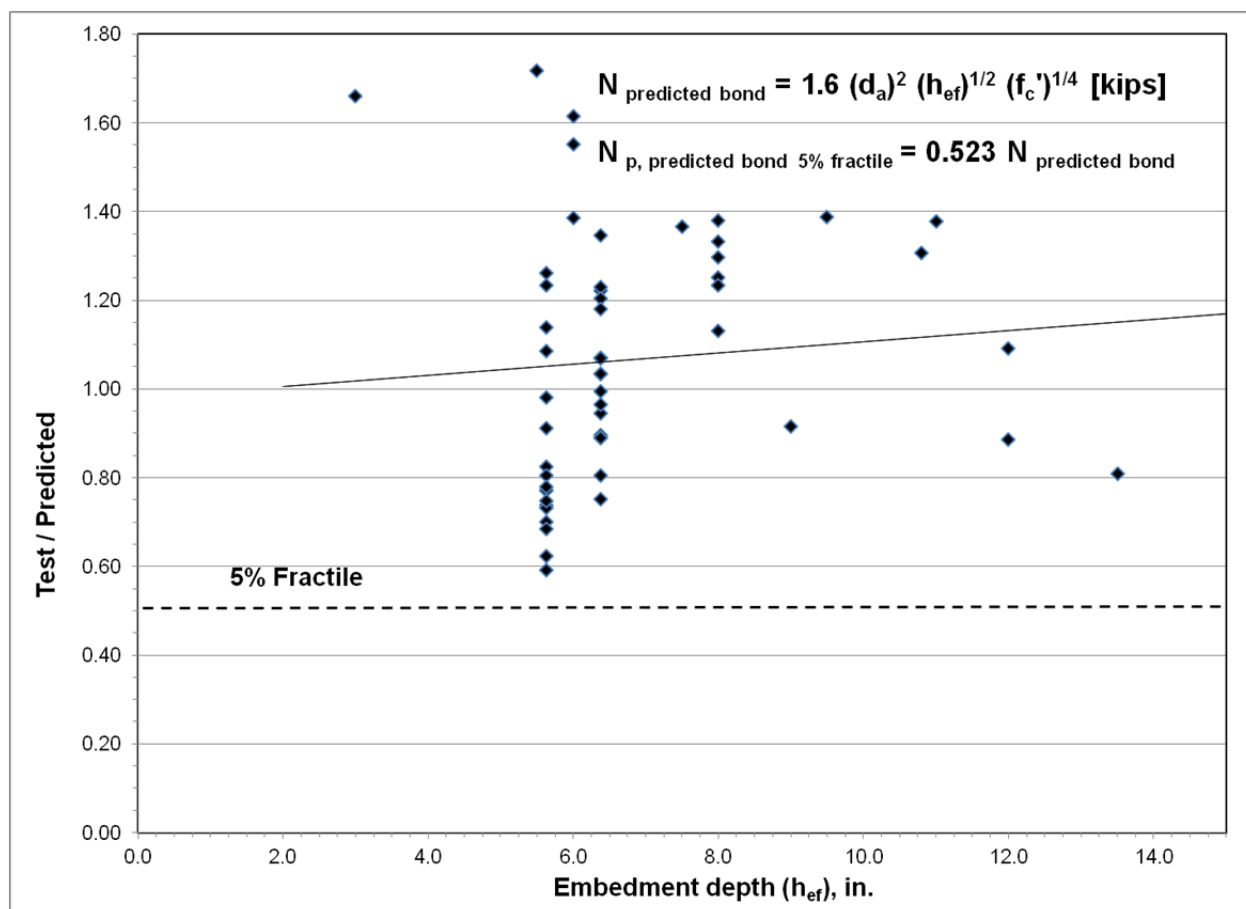


Figure 9b - Simplified average and 5 percent fractile design equation - bond exponential model.

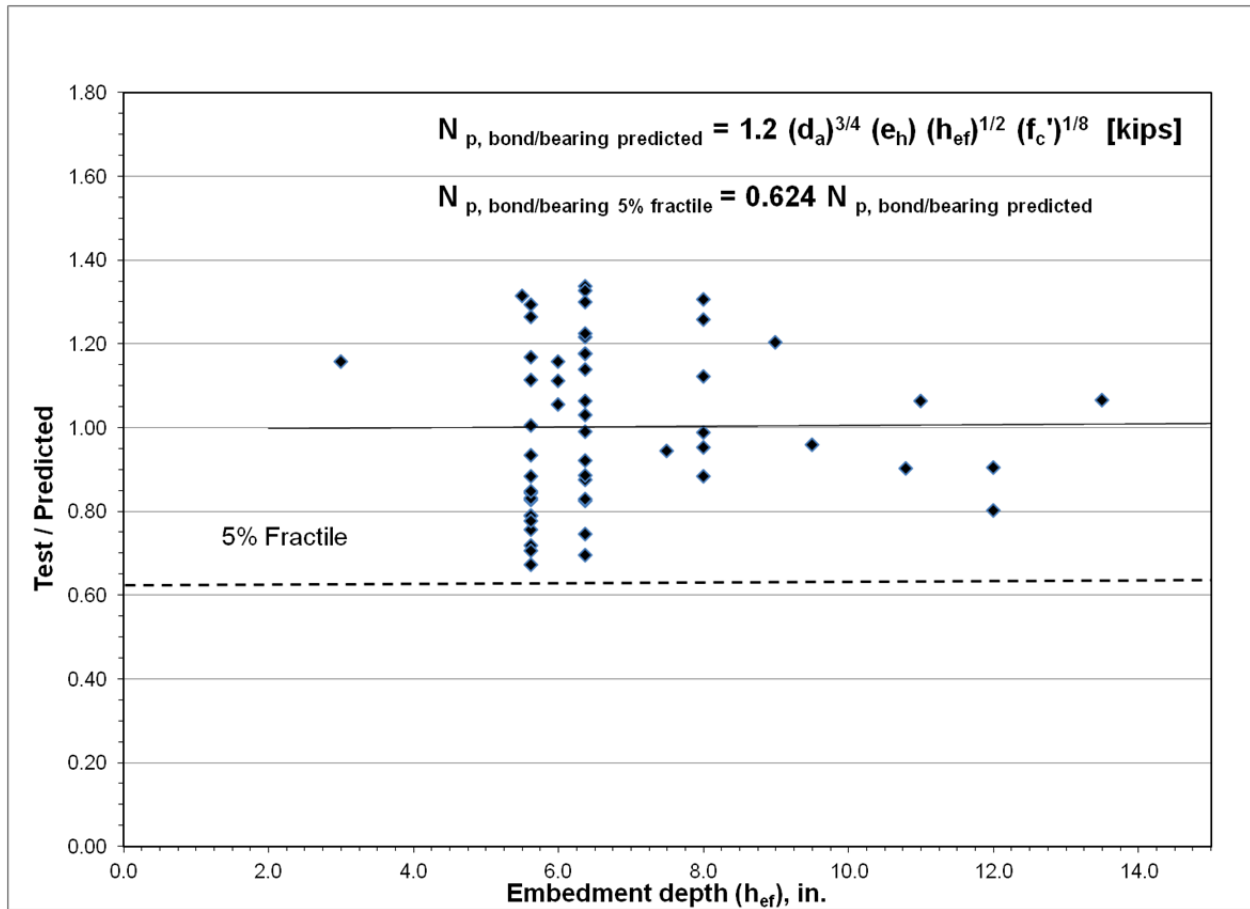


Figure 9c - Simplified average and 5 percent fractile design equation - bond/bearing exponential model.

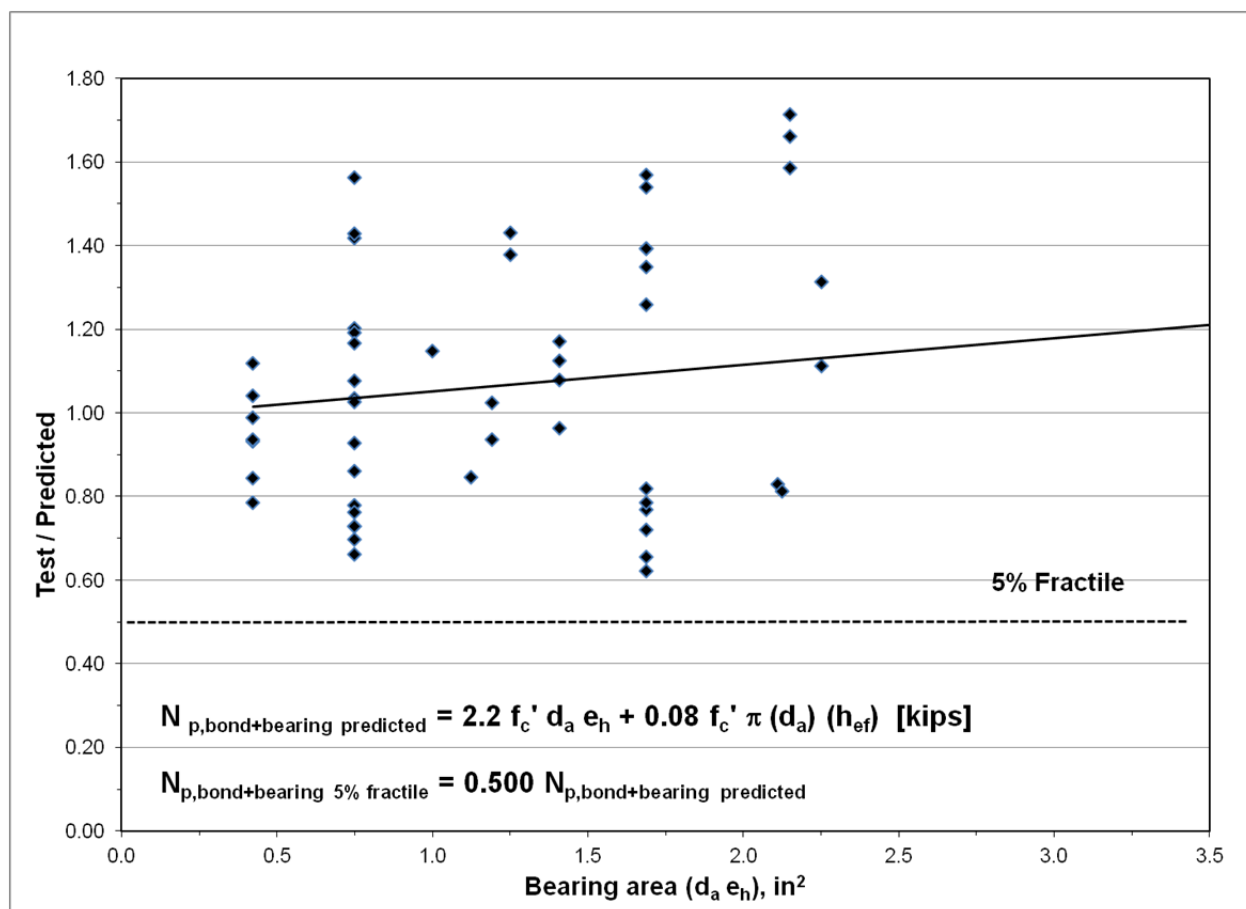


Figure 9d - Simplified average and 5 percent fractile design equation - bond+bearing addition model.

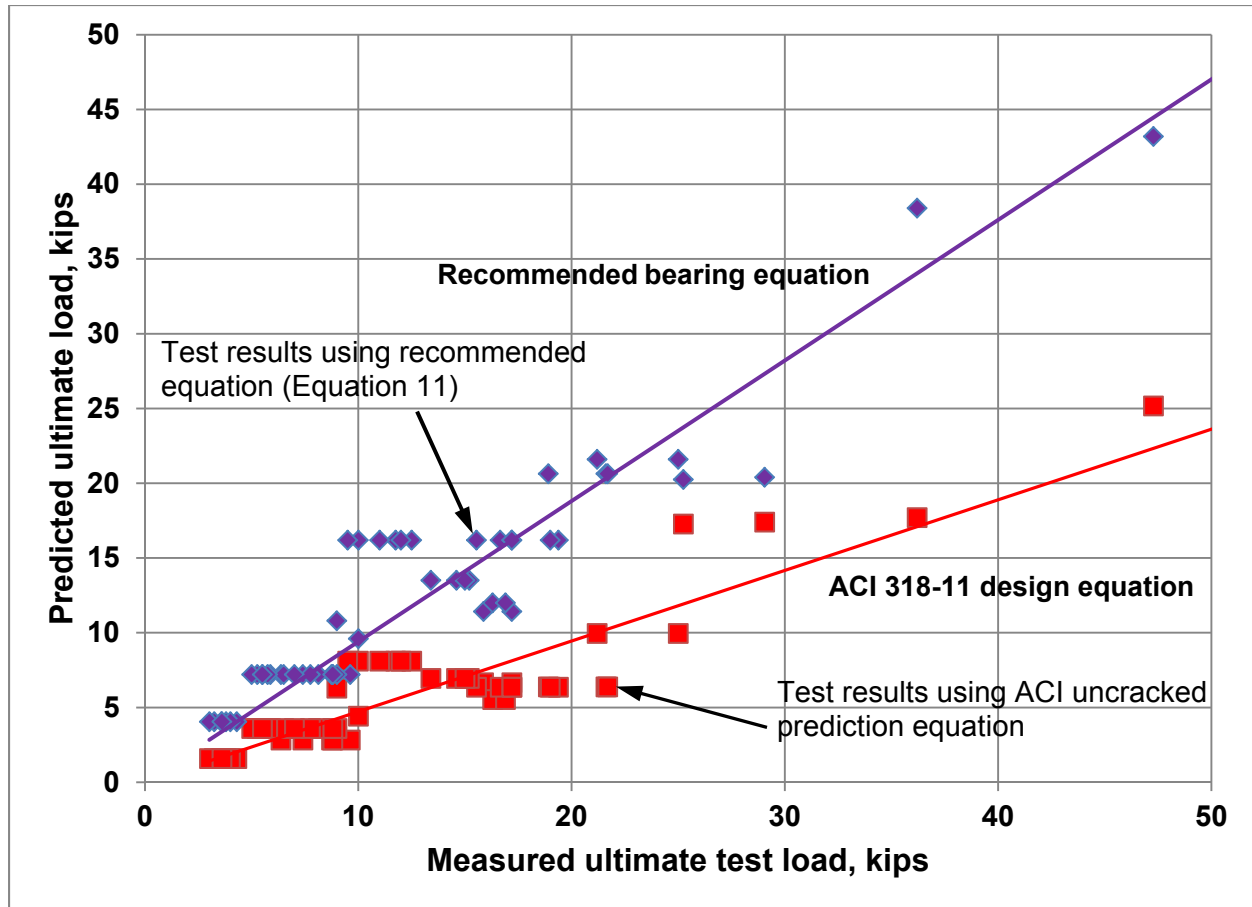


Figure 10 - Plot showing comparison of measured uncracked test load to predicted uncracked exponential bearing load.

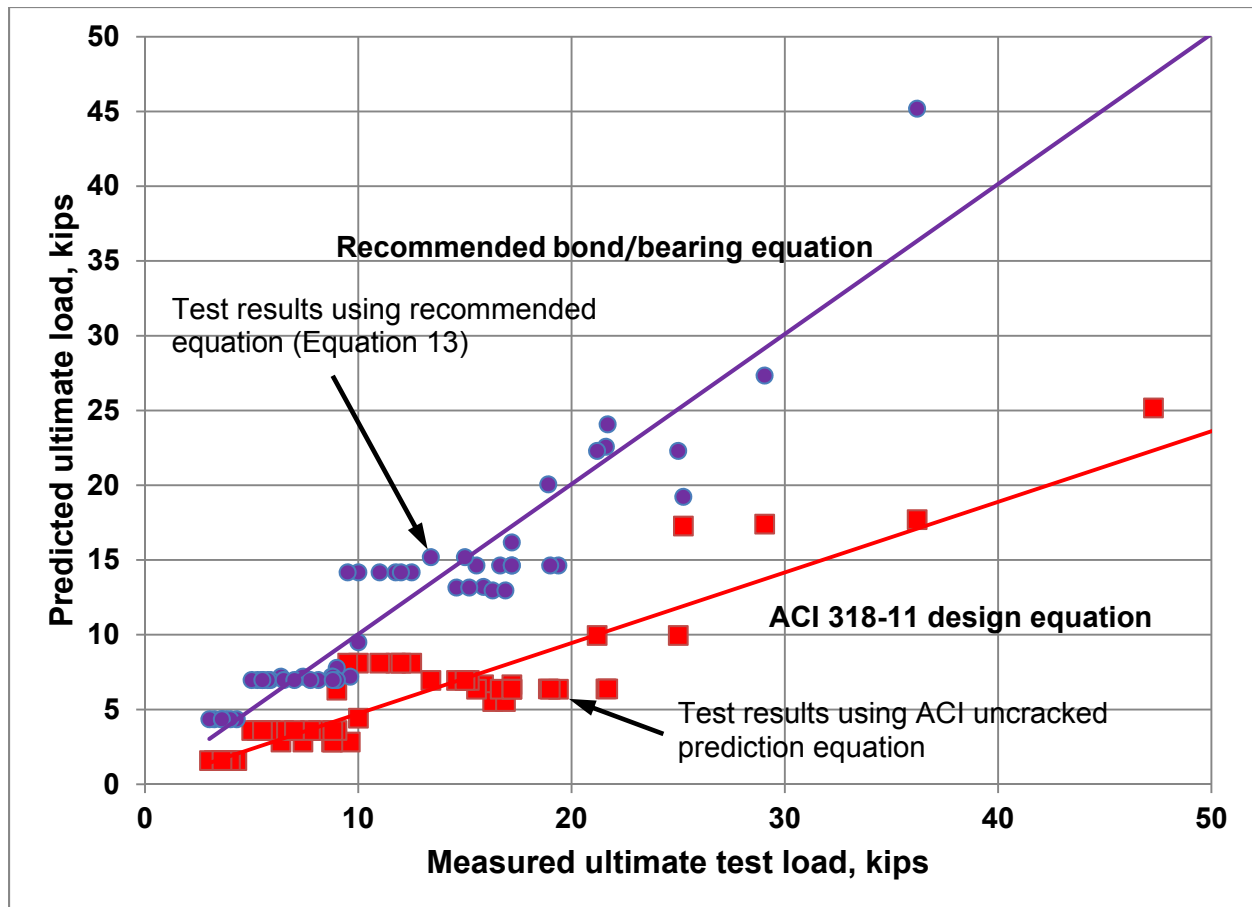


Figure 11 - Plot showing comparison of measured uncracked test load to predicted uncracked exponential bond/bearing load.

PERFORMANCE OF BEAM-COLUMN JOINTS IN THE 2010-2012 CHRISTCHURCH EARTHQUAKES

Roberto T. Leon
Weng Yuen Kam
Stefano Pampanin

Synopsis: The design of beam-column joints in reinforced concrete moment frames is an area where USA and New Zealand standards have diverged for many years. USA design guidelines, and ACI 352 in particular, implicitly accept damage in the form of shear cracking, bar slip and possible column hinging for joints subjected to large lateral load reversals. Since the 1980's, the New Zealand approach has been to minimize that type of damage and to concentrate the deformations in plastic hinges in the beams by careful detailing of the joint and adjacent beam regions, thus keeping columns essentially elastic. The recent February 22, 2011 Christchurch earthquake and its associated swarm present an excellent opportunity to contrast these approaches in terms of visual performance for a variety of New Zealand structures detailed and built before and after the newer, more stringent joint design guidelines came into effect. The main lesson from the Christchurch experience is the importance of providing both some degree of lateral resistance, e.g. via beam-column joint moment-resisting capacity, and an increased level of displacement capacity in secondary or gravity-frames in order to improve the overall building's robustness and seismic resilience in response to earthquake demands beyond the code design level.

Keywords: beam-column joints, joint design, moment resisting frames, seismic damage, seismic performance.

Roberto T. Leon, FASCE, FACI received his Ph.D. from UT-Austin in 1983 and is the D.H. Burrows Professor of Engineering at Virginia Tech. He is a long-time member of ACI 352 (Monolithic Joints) and was in Christchurch, NZ, as an Erskine Fellow at the University of Canterbury when the February 22, 2011 earthquake occurred.

Weng Yuen Kam, MACI, received his Ph.D. from the University of Canterbury in 2011 for developing techniques for seismic assessment and retrofit of non-ductile reinforced concrete beam-column joints. He is a Senior Structural Engineer with the Christchurch office of Beca Carter Hollings & Ferner Ltd., Auckland, New Zealand.

Stefano Pampanin, MACI, is Reader and Chair of the Structural/Geotechnical Cluster in the Department of Civil and Natural Resources Engineering at the University of Canterbury, New Zealand. He is President of the New Zealand Society for Earthquake Engineering (2012-2014) and was appointed by the Department of Building and Housing as member of the Expert Panel to assist with the technical investigation into the performance of critical reinforced concrete buildings following the 22 Feb 2011 earthquake.

INTRODUCTION

The city of Christchurch, New Zealand, is intimately tied to the development of seismic design provisions for reinforced concrete structures through the work of many researchers at the University of Canterbury (UoC) since the late 1960s. Along with others (Bertero and Mahin, 1977), the work of UoC Profs. R. Park and T. Paulay and their collaborators (Park and Paulay, 1975) put the concepts of capacity design and ductility at the forefront of worldwide discussions on design approaches for reinforced concrete frame and wall structures.

The critical role of connection performance in moment frames was clearly established through the early attempts to develop mechanistic frameworks for beam-column joint shear behavior, which led to a basic agreement on design approaches based on truss and diagonal strut mechanisms (Hanson and Conner, 1967; Corley and Hanson, 1969; Park and Paulay, 1973; Meinheit and Jirsa, 1981; Leon and Jirsa, 1986). However, an early multi-national (USA-Japan-NZ-China) research project (Cheung et al. 1991; Jirsa: 1911, ACI352R, 2002) failed to reach an agreement on the amount of the joint shear that should be apportioned to each mechanism and on the relative importance of factors such as slab effects, bidirectional loading, axial loads and confinement on joint behavior; this divergence was predominantly due to differences in interpretation of test results and acceptable performance criteria (Jirsa, 1991).

USA design documents, and the original ACI 352 report in particular (ACI 352, 1985), implicitly accept a high level of damage in the form of shear cracking, bar slip and possible column flexural hinging for joints in moment frames subjected to large lateral load reversals. There have been several updates to the ACI 352 original document (ACI 352R, 2002) that have added additional qualifications, provisions and commentary to the original version, but the basic design values have not changed significantly. The New Zealand approach, since the mid-1980s (NZS3101, 1982) has been to minimize that type of damage and concentrate the deformations in plastic hinges in the beams by careful detailing of the joint region and explicit capacity-design of the joint and columns. The recent Christchurch earthquakes present an excellent opportunity to contrast these approaches.

This paper first presents a short description of the series of earthquakes that hit the Canterbury area over the last two years to highlight some important ground motion characteristics and local site effects that may have contributed to the observed performance. It then briefly describes NZ reinforced concrete construction pre- and post-1982 when the new design provisions began to be extensively used and gives a brief comparison of USA and NZ beam-column joint design. Finally, the performance of beam-column joints on some typical buildings is summarized and conclusions on the observed vs. expected performance are given.

THE CHRISTCHURCH 2010-2012 EARTHQUAKES

The city of Christchurch lies on the Canterbury Plain, in the Eastern central coast of the South Island of New Zealand. Until 2010, the main seismic risk for the city was considered to come from the Alpine Fault that runs along the western coast of the island at a distance of about 130 km (80 m.). Thus design accelerations for seismic design in Christchurch ($PGA=0.22g$, with an elastic seismic coefficient for very soft soil site, and low-period structures, class D, $C_v=0.66g$) were comparable to those in an old UBC Zone 3 in the USA (about $0.54g$) for the plateau of the spectrum. Beginning in September 2010 and continuing through today, ground movements in a series

of blind fault(s) have led to a swarm of large earthquakes that have devastated the central business district (CBD) and much of the eastern suburbs of Christchurch. Among the more than 10,000 tremors felt during this period are the following main events shown in Figure 1:

- Sept 4, 2010 (Darfield Earthquake): a M_w 7.1 complex strike-slip rupture at a distance of about 40 km (25 m.) and focal depth of about 10 km (6 m.). This event was about a MMI 7-9 in the Christchurch CBD, with a strong-motion duration of 8-15 seconds and estimated to have generated spectral acceleration on the order of 70% to 90% of the design level earthquake (DLE) for Christchurch CBD.
- Dec. 26, 2010 (Boxing Day): a M_w 4.7 strike-slip rupture at about 2 km (1.3 m.) distance and 5 km (3 m.) focal depth. This event resulted in damage consistent with a MMI 7-8 in CBD given its epicentral location, with a duration of about 5 seconds duration and spectral accelerations on the order of 40-60% of DLE.
- Feb. 22, 2011 (Christchurch Earthquake): a M_w 6.2 oblique-reverse rupture at about 10 km (6 m.) distance and 5 km (3 m.) focal depth. Damage in the CBD was consistent with a MMI 9-10, about 8-10 seconds duration, and spectral accelerations in the order of at 150% to more than 250% of the DLE.
- June 13, 2011: a M_w 6.0 oblique-reverse rupture at about 5 km (3 m.) distance and 5 km (3 m.) depth. Surveys indicate damage of about MMI 7-9, about 6-8 seconds duration, and accelerations of about 90% DLE.
- December 23, 2011: a M_w 6.0 oblique-reverse rupture at about 5 km distance (3 m.) and 13-15 km (8 to 9.5 m.) depth. This event resulted in about MMI 7-8 effects, with duration of 13-15 seconds, and accelerations of about 50% of DLE.
- December 23, 2011: a M_w 6.0 oblique-reverse rupture at about 5 km (3 m.) distance and 10 km (6 m.) depth (about MM 7-9) about 10-12 seconds duration, and accelerations of about 60% of DLE.

The duration given above is defined by the approximate length of record containing horizontal accelerations over 0.1g within the CBD recording stations. The distance generally refers to the distance to the fault trace except for the Dec. 26, 2010 and Dec. 23, 2011 earthquakes, for which the distance to earthquake hypocenter is given.

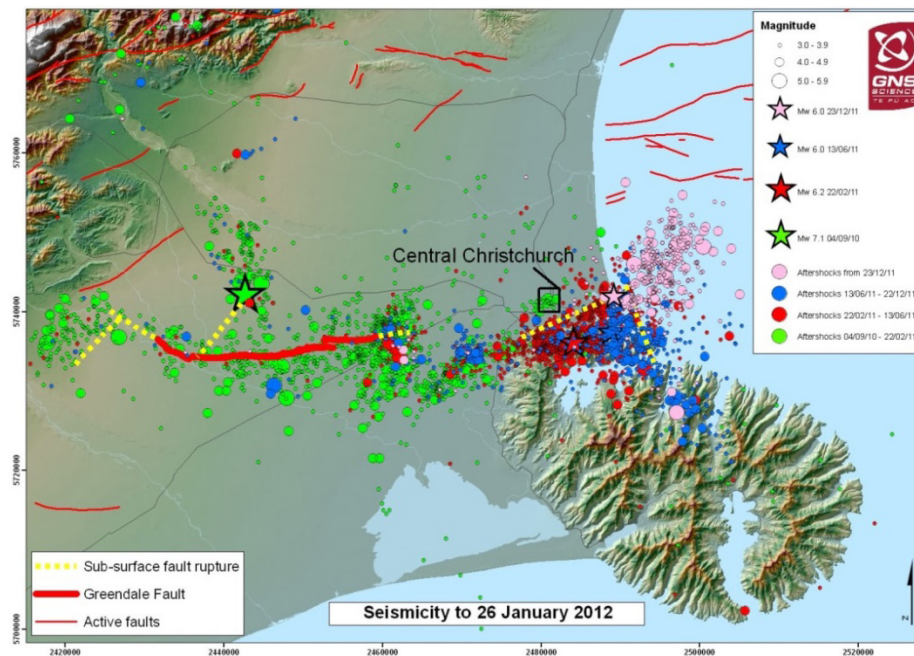


Figure 1 - Main events in the Canterbury Earthquake Swarm of 2010-2012 (GNS, 2011)

This series of earthquakes and in particular the very damaging February 22, 2011 event, share some important and unusual ground motion characteristics that need to be considered when assessing building performance:

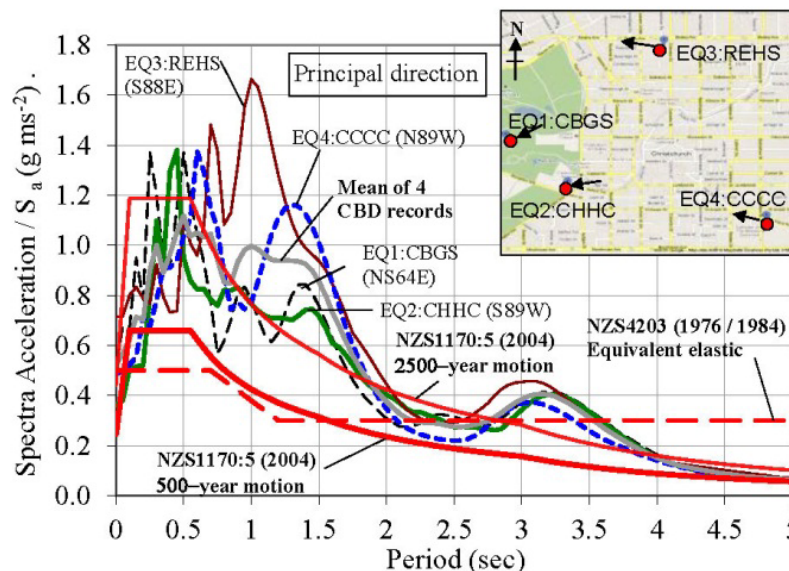
- Shallow (5 to 10 km; 3 to 6 m.) and very close to the CBD (<10 km or <6 m.) hypocenters, thus being strong, near-field events perhaps comparable only to those that affected Agadir (Morocco) in 1960, Managua (Nicaragua) in 1972 and Chi-Chi (Taiwan) in 1999, events where the damaged city was located directly above or very close to the epicenter.

- Large vertical accelerations with high frequency content, including one station located directly above the epicenter with peaks above 2g; spectra for these accelerations show a large peak around the 0.1sec period and decrease rapidly after the 0.2 sec limit.
- Horizontal accelerations in the CBD for the February 22, 2010 event in one direction were in excess of 180% of design code event (DLE); accelerations in at least one other event were close to 100% of the DLE.
- Relatively short duration for the individual events, but cumulatively in the range of 1 minute of strong motion.
- Significant evidence of near-source effects, topography and basin effects, and near-surface nonlinearities in the soft glacial deposited soils in many of the strong ground motion recorded. The response spectra of the records show a clear amplification in the 2 sec to 2.5 sec period range.
- Widespread manifestation of surface liquefaction and lateral spreading; and in parts of the CBD, significant building differential settlements were observed.

Extensive discussion of these particular issues is beyond the topic of this paper. Readers are referred to the reports on seismicity and ground motion for the Canterbury Earthquake Royal Commission for more details of the seismological (Bradley, 2011) and ground deformation phenomena (Cubrinovski et al., 2011), as well as to the Special Issues of the Bulletin of the NZNSEE dedicated to this earthquake (NZSEE, 2011).

A good estimate of the intensity of ground motion can be gleaned from the response spectra derived from the records of the principal horizontal motions at four stations around the CBD. Figure 2 shows the data for the four stations, their average, and the 475-year and 2475-year design spectra for central Christchurch. It is clear that the February 22, 2011 event was well beyond the DLE and those structures with a fundamental period in the range of about 1 to 2 seconds (frames in the 10 to 20 story range by conventional wisdom) were particularly hard hit. In addition, the figure highlights the variability of records obtained in a relatively small area; all four stations were within 1.5 km (1 m.) of the center of the CBD and the large differences are probably due to local soil conditions.

Figure 2 - Spectra from 4 stations in the CBD (Kam 2011)



Further description of the intensity of ground motions experienced in the Christchurch CBD, and how these compare to those used in design of new buildings prior to 22 February 2011, and the general overview of the structural performance of buildings of different types during the 2010-2011 Canterbury earthquakes is available in a report published elsewhere (Kam and Pampanin, 2011).

CHRISTCHURCH BUILDING STOCK AND NZ SEISMIC DESIGN PROVISIONS

The building stock in the Central Business District (CBD) of Christchurch was similar to that of many Midwestern and Eastern cities in the USA. There was a very large number of low-rise commercial and residential unreinforced masonry (URM) structures predating the early 1930s, when the Hawkes Bay earthquakes clearly exposed the vulnerability of this type of construction in New Zealand. The Hawkes Bay earthquakes led to the first "Draft