THE FAUSTIAN AND THE MAGIAN
IN THE BUILDING OF A BUILDING CODE

by

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Introduction

This is the best of times and the worst of times to commit to a single perspective on the nature of building codes. It is the best of times because of the existence of massive tested knowledge on structural theory. It is the worst of times also because of the existence of massive tested knowledge on structural theory. The enormity of the pool of knowledge enables and disables the development of consistent goals and methods of operation for assembling workable building codes.

The art and the science of building have evolved through centuries in a state of tension between the Faustian, the use of science and technology to overcome the limits of the physical world, and the Magian, reliance on the magic of phenomenology, approaches. To develop a new code or to improve an existing one, it is essential to appreciate this tension. To understand the essential characteristics of a building code, we start with four questions often asked about building codes: “Why? How? Who? and What?”

The least difficult one to answer is “why?”

Engineering is a profession. It involves the use of judgment. There are indeed many engineering decisions that appear to be made within the bounds of the supporting sciences. In those cases, there is relatively little reason to limit the prerogatives of the engineer. But the supporting sciences seldom provide crisp answers to engineering problems. To build, the engineer has to use judgment. Judgment has soft boundaries and is influenced strongly by what the engineer considers to be acceptable risk. To reduce the probability of innocent or deliberate misuse of judgment, the wise of the profession need to come together and place bounds on the use of judgment. Cross (Cross, 1952) answered the question “why” succinctly by suggesting the building code as an effort “to keep out the fools and the rascals.”

It is difficult to separate the answers to the questions “who? and “how?” Certainly, the code should be developed by a group of engineers who have experience and have demonstrated good judgment in their careers. But who should set up the organization? Should it be a government body with authority and yet without responsibility? Or should

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Simple as it may appear, to respond to the question “what” is the most difficult of the four. Well-meaning, intelligent, and experienced professionals take extreme and often contradictory positions on this issue. The arguments circle around the three principles stated by Vitruvius that are, in short, (1) Strength, (2) Serviceability, and (3) Beauty. Some assert strongly that, in a free society, the code should be limited to concerns about public safety and focus only on strength. Others are equally firm in their opinion that the built environment is, ultimately, communal property and the code should also address serviceability though they stop short of beauty that would seem to go with serviceability. The arguments are also layered by the options of holding the engineer responsible at the time of design (prescriptive code) or only if a problem arises after construction (performance-based code).

A building code is a social contract. It is out of place to frame it completely in a rational domain. In a world where more important issues are settled by fundamentally different approaches such as Napoleonic Code and Common Law, to hold a thesis for a particular flavor of building code is not constructive. The building code is to shape human experience in building and is shaped by human experience in building. Unfortunately there is no universal first step and there is no unique format. There is, however, the same tension in all building codes of any type between science and engineering practice caused by the fact that, in the practice of engineering, the choice is not between right and wrong but between good and bad. It is not impossible for the right to be bad and for the wrong to be good.

The following three sections represent an attempt to illustrate the tension between the Faustian and the Magian approaches in building.
Definition of Static Moment for Flat Slabs

In engineering jargon of the USA, a reinforced concrete flat slab is a slab supported directly on columns. By definition, a flat slab requires a column capital (Fig. 1) and sometimes a drop panel (a portion of the slab near the column made thicker by “dropping” or lowering the appropriate panels of the formwork). Without the panel and the capital, it is called a flat plate (Fig. 2).

The events in the development of an expression for the static design moment, $M_o$, (absolute sum of the moments in a given direction acting at the column centerline and at mid-span) for a typical interior panel followed an interesting course that illustrates the stark difference between engineering practice and science.

Even though it may seem like a natural for reinforced concrete construction, the flat slab had to be invented because of the dominance in classical times of the beam-and-girder construction. The load was carried by the planks to the beams, by the beams to the girders, by the girders to the columns, and then to the foundations. The conceptual construct was that the load traveled in lines perpendicular to the supporting element. In that frame of mind, the “two-way slab,” or a slab supported by girders on all four sides seemed to be the natural solution (Fig. 3). And it took an enterprising contractor, C.A. P.
Turner, who was bothered by the interference of the girders with utility lines and the additional cost of formwork, to build a reinforced concrete structure without any girders in 1906.

Figure 3

What Turner accomplished was not beyond physics but it was well beyond the engineering science of the time. That it did not fit into the conventional wisdom did not stop the contractors all over the world from adopting the idea rapidly. By 1913, over one thousand flat slabs had been built using a cacophony of design methods. The confusion in concept is readily illustrated in Fig.4 showing the amount of reinforcement used in a typical interior panel with 20-ft spans. Clearly, engineering practice had transcended engineering science. There had to be a meeting of minds to accommodate the hard facts (the structures built served well) over and above the science of the day.

To add to the heat of the debate that had already been initiated on the premise that the flat slab was “beyond the range of pure analysis,” a young engineer from Boston, J. R. Nichols, produced a paper, “Statistical Limitations upon The Steel Requirement in Reinforced Concrete Flat Slab Floors,” (Nichols, 1914) in which he argued that the absolute sum of the moments acting on the free-body diagram including one half of a typical interior panel (Fig.5)\(^2\) could be determined by simple statics.

\(^2\) An “interior panel” is defined as one bounded on all sides by similar panels loaded similarly so that the column centerlines and panel centerlines may be considered to be lines of symmetry.
Recognizing that, as a result of symmetry, there would be no twisting moments and shears along the boundaries of the free body considered, Nichols arrived at Eq. 1 which did not identify the quantitative relation between $M_1$ and $M_2$ but simply stated that their absolute sum should not be less than the static moment, $wL_2L_1^2/8$. 


\[ M_1 + M_2 = M_o = \frac{w \cdot L_2 \cdot L_1^2}{8} \]  

(1)

\[ M_1 = \text{Total moment acting on a panel at column centerline (negative moment)} \]

\[ M_2 = \text{Total moment acting on a panel at mid-span.} \]

\[ M_o = \text{Total static moment} \]

\[ w = \text{Uniform load} \]

\[ L_1 = \text{span length center-to-center of columns in the direction moment is being computed} \]

\[ L_2 = \text{length of transverse span} \]

To project the basic idea to a practical case, Nichols considered a typical rectangular interior panel supported by round capitals to obtain

\[ M_o = \frac{wL_2 \cdot L_1^2}{8} \left[ 1 - \frac{4c}{\pi L_1} + \frac{1}{3} \left( \frac{c}{L_1} \right)^3 \right] \]  

(2)

\[ c = \text{outer diameter of the column capital} \]

Later, Nichols simplified Eq. 2 to read

\[ M_o = \frac{wL_2 \cdot L_1^2}{8} \left( 1 - \frac{2c}{3L_1} \right)^2 \]  

(3)

It was a simple statement based on respectable mechanics but it turned out to be anathema to a good segment of the building profession. It required more reinforcement than many were providing.

Strain measurements (Lord, 1910) in tests of flat slabs were used to test Nichols’s assertion. Surprisingly, the verdict was against Nichols. It was an exemplary case of the conflict between analysis and design methods. At that time, the straight-line formula was used in design to set the relationship between normal stress and moment. It was based on the assumptions that there was a linear relationship between stress and strain, that the
strain distribution over the height of the section was linear, and that the tensile strength of the concrete was zero.

\[
M = A_s f_s j d = A_s \varepsilon_s E_s j d
\]  

(4)

\(M = \) resisting moment  
\(A_s = \) cross-sectional area of reinforcement  
\(jd = \) internal lever arm with \(j \) equal approximately to \(7/8\)  
\(f_s = \) unit stress in longitudinal reinforcement  
\(\varepsilon_s = \) unit strain in longitudinal reinforcement  
\(E_s = \) Young’s modulus for reinforcement

It worked elegantly and safely for proportioning reinforced concrete sections. Ignoring the tensile strength is an appropriate assumption for proportioning sections but is not appropriate for interpreting data. In the tests conducted, the applied load generated a unit moment \(m_a\) on a slab section (Fig. 6). The strain developed was \(\varepsilon_b\) because part of the moment was resisted by concrete in tension and was less than strain \(\varepsilon_a\) that would have been developed had the section been fully cracked. In interpreting the test data, the reinforcement stress was based on the measured strain \(\varepsilon_b\). The moment determined using Eq. 4 and the strain \(\varepsilon_b\) was a fictitious moment \(m_b\) smaller than the applied moment \(m_a\). This misinterpretation, based on the conventional wisdom of the times, provided ammunition for those who did not believe statics would apply to slabs without beams. The conceptual vision that the load in a flat slab did not travel in only one direction at a time, as in traditional construction, but scattered like buckshot led to the misjudgment that statics did not apply.
The 1920 version of the ACI Building Code (ACI Committee, 1920) used Eq. 5 to define static moment in a typical interior panel

\[ M_0 = 0.09 \cdot w \cdot L_2 \cdot L_1^2 \left( 1 - \frac{qc}{L_1} \right)^2 \]

\[ \text{Eq. 5} \]

q was defined to be 2/3 for round capitals and ¾ for square capitals. The judgment of the committee members writing the code appeared to have been influenced by Nichols’s style but not by Archimedes.

In 1921, the treatise on reinforced concrete slabs by Westergaard and Slater consummated the compromise between statics and conventional wisdom. The Joint Committee (Joint Committee, 1925), convened to revise the code, adopted a modified version of Eq. 4

\[ M_0 = 0.09 \cdot w \cdot L_1 \cdot L_2^2 \left( 1 - \frac{2c}{3L_1} \right)^2 \]

\[ \text{Eq. 6} \]

adding the following interesting statement “The sum of negative and positive moments provided for by this equation is about 72% of the moment found by rigid [sic] analysis based upon the principles of mechanics. Extensive tests and experience with existing structures have shown that the requirements here stated will give adequate strength.”

The events leading to the statement above and the statement itself illustrate clearly the distinction between engineering and science. Let us retrace the steps. Turner builds a flat slab without analysis. It works. Others build similar structures but with varying amounts of reinforcement. One engineer, Nichols, shows the static requirement, or the static moment \( M_0 \), for the absolute sum of the moments at the critical sections. The static moment, in accordance with the accepted relationship between moment and permissible reinforcement stress demands a minimum amount of longitudinal reinforcement. The sitting structures, as interpreted by the analysis methods used, defy Nichols. So does interpretation of the observed reinforcement strains. Finally Westergaard and Slater show that there is no conflict between measurements and statics. The wise persons of the profession meet again. They seem to recognize that Nichols was right theoretically. However, they are not convinced the sitting structures are wrong. Given the choice, they side with experience and deny science. They choose the good, as they perceive it, over the right. It is important to note that their statement of “72%” of the static moment suggests they thought Nichols’s analysis was right.

It took until 1971 (ACI Committee 318, 1971) to right the wrong and substitute 0.125 for 0.09. Even then, the flat slab designed for a given load did not change much in
proportions and amount of reinforcement. The safety factors and methods for calculating resistance changed to maintain the product more or less as it was designed using the incorrect definition of the static moment.

**Web Reinforcement**

The conflict for the definition of the design static moment was between well understood physics and poorly understood experience. The conflict in the proportioning of web reinforcement in beams was and continues to be poorly understood physics and reasonably well understood experience.

To keep the discussion simple, we shall focus on “vertical” web reinforcement (perpendicular to the longitudinal reinforcement) in reinforced concrete beams under monotonically increasing shear. And we shall stay away from questions concerning beams without web reinforcement.

The first all-inclusive vision on design of web reinforcement in the USA may be ascribed to Richart (Richart, 1927). It is relevant to note that he started his experimental investigation in the early 1900’s but could not bring himself to publish a full report until 1927. In retrospect, we may presume that, in an age of enlightenment, he was seeking a compact and crisp solution that never came to be. It is not an exaggeration to claim that anyone else working on web reinforcement, before and after Richart, started or ended with variations on the theme of the truss analogy by Mörsch.

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**Figure 7**

*Compression Diagonal*  
*Top Chord*  
*Web Member (in tension)*  
*Tension Chord*
Once one makes the egotistical imposition (it is not quite proper to call it an assumption) on the beam and call it a truss, things fall into place (Fig. 7). Considering the equilibrium of a joint with the forces shown and an imaginary strut conveniently at 45 deg. with the horizontal, we arrive at the “rigorous” result that

\[
A_w f_w \cdot \frac{d}{s} = V
\]

(7)  

*Aw* = cross-sectional area of all legs of one stirrup or hoop used as web reinforcement  

\(f_w\) = unit stress in stirrup  

\(d\) = effective depth of prismatic beam  

\(s\) = spacing of stirrups.  

\(V\) = Applied total shear force

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3 Broken lines in the figure represent expectations based on the truss analogy. The figure is reproduced from Richart, 1927.
Using Eq.7 as a way to understand the experimental results, Richart ran up against a wall. Theory demanded that all shear be resisted by the web reinforcement. Test results indicated consistently otherwise (Fig.8). The conundrum was similar to that for the flat slab, if he followed “rigid” theory the contractors would get up in arms. And if Richart abrogated one part of the derivation, all other parts would become questionable. For example, he referred as plausible to the expression for shear strength derived earlier by Slater et al (Slater 1926)

\[
(0.005 + \rho_w)f_w = \frac{V}{b \cdot j \cdot d}
\]  

\(\rho_w = \frac{A_w}{bs}\), web reinforcement ratio (for vertical stirrups) where b is the width of the compression flange and s is the stirrup spacing. 
\(jd = \) internal lever arm based on the straightline formula. 
\(b = \) width of compression flange 
\(f_w = \) unit stress in vertical web reinforcement 
\(V = \) total applied shear force

Strictly, the very recognition of Eq. 8 stood the truss analogy on its head. The expression implied the existence of a shear resistance \((0.0005 f_w)\) which appeared to have nothing with the amount of web reinforcement provided. Ergo, the truss analogy was irrelevant.

Eventually, Richart went against his analytical credo and suggested the form

\[
\frac{V}{b \cdot j \cdot d} = C + \rho_w \cdot f_w
\]  

\(V = \) total applied shear force 
\(C = \) a constant stress related to compressive strength of concrete such as 0.03 \(f'_c\), where \(f'_c\) is the cylinder strength. 
\(b = \) width of compression flange 
\(jd = \) internal lever arm based on the “straightline formula” 
\(f_w = \) unit stress in vertical web reinforcement

It is important to note that Eq. 9 refers to working stresses. It is even more important to quote his last conclusion (Richart, 1927):
For descriptions and discussions bearing on the details of the action of web reinforcement, the text may well be consulted, and the information there contained may be expected to aid in comprehending the behavior of the reinforced concrete beam under a variety of conditions and to give knowledge that may be useful in design and construction – matters that cannot be summarized here.

The last clause is noteworthy and admirable. Richart was admitting that he did not have a complete handle on the problem. He had started with the truss analogy and ended up with a form that abrogated the truss analogy. Nevertheless, the second term on the right-hand side of Eq. 9 was interpreted in terms of the truss analogy. It was sheer contradiction. But epistemological inconsistency was of little concern to the code writers. Richart’s half-theory was considered to be better than none and accepted to remain in that form until the 1960’s in the ACI Building Code.

Until the 1950’s, research was seldom a full-time job for a structural engineer. Advances in the field were typically made by practicing engineers, such as Turner and Nichols, or gifted instructors who could devote part of their time to investigating problems of interest to the profession, such as Westergaard and Richart. World War II changed the scene in structural engineering as it did in many other segments of society. There developed a small coterie of almost full time researchers. A few years after WWII, they were looking around for a cause. They found it in the series of shear failures experienced by reinforced concrete warehouse frames (Anderson, 1957). His kick-off statement is still valid:

All engineers interested in concrete construction are aware of the abundance of excellent research work being conducted in all phases of concrete technology. Engineers are also equally familiar with a variety of experience showing that concrete has been a most reliable and economical engineering material. Nevertheless, in spite of these extensive research programs and in spite of the successful use of concrete in structures of all types, the fact remains that the fundamental behavior of concrete members in transferring shear load, one of the most common design features, is not fully understood, and in certain specific cases design practice has been used that has resulted in costly failures.

The warehouses had been designed in accordance with the ACI Building Code. There had to be something lacking in the code requirements developed by Richart and fellow committee members. A joint committee of the American Concrete Institute and of the American Society of Civil Engineers was formed. They sponsored tests to investigate the problem (Elstner) and reported their findings in 1963.
Some of the principles they found appropriate for the shear problem are worth quoting in part:

1. **Diagonal tension is a combined-stress problem.**
2. **Failure may occur immediately on formation of a diagonal-tension crack or at a higher load.**
3. **The shear corresponding to the formation of the diagonal-tension crack, ignoring any possible increase, defines the strength of the beam without web reinforcement.**
4. **Distributions of shear and flexural stresses over the cross section of a reinforced concrete beam are not known.**

Principle 4 did not stop the committee members from using mechanics to determine principal tensile stresses to come up with the conclusion that the main parameters affecting the unit shear strength of a reinforced concrete beam (defined arbitrarily as \( v_u = V_u / bd \) for a rectangular section where \( v_u \) is the unit shear strength, \( V_u \) is the shear strength, \( b \) is the width of the compression flange and \( d \) is the effective depth) without web reinforcement where \( M/Vd \) (ratio of moment to the product of the shear force and the effective depth which can be represented by \( a/d \) in beams with concentrated loads where \( a \) is the length of the shear span), \( \rho \) (longitudinal reinforcement ratio, \( A_s / bd \) ), and the tensile strength of the concrete assumed to be proportional to \( \sqrt{f_c^*} \). Faced with the squalor of the experimental results, the final expression was, de facto,

\[
v_c = 2 \cdot \sqrt{f_c^*}
\]  

(10) although a longer form including \( \rho \) and \( M/Vd \) continues to grace the ACI building code. Suspicions about size effect were relegated to the background. The effect of longitudinal reinforcement was virtually effaced by lumping it with the factor \( M/Vd \). The main issue was, of course, the selection of web reinforcement. Reference was made to the then-current expression in the code

\[
v = \frac{V}{bjd} = 0.03f_c^* + K \cdot \rho_w \cdot f_w
\]  

(11) Which was the Magian compromise with the factor \( K \) derived from the truss analogy.\(^4\) The committee members observed (ACI ASCE Committee 326,1960)

\[^4\] K=(\sin \alpha + \cos \alpha \cdot \sin \alpha \) where \( \alpha \) is the angle between the web reinforcement and the horizontal.
Although the nature of this design procedure has not been changed significantly since 1921, the limitations and maximum values have been altered in nearly every revision of the ACI Code. Like the basic design principles, the limitations and maximum values have tended to be products of logical reasoning rather than systematic laboratory tests.

The implication is clear. Tests are better than thinking. And the tests need to be done under laboratory control. They continued:

*In general, the design criteria for web reinforcement of the 1956 ACI code were not developed directly from laboratory tests. They are not in full accord with our basic understanding of the function of web reinforcement and the mechanisms of failure. However, it is a fact that the design criteria have withstood the test of time. No structural failures known to Committee 326 can be directly traced to possible inadequacies of these design provisions for members with web reinforcement.*

The Magian had again triumphed over the Faustian. We are indebted to the committee for reminding us once again that a design algorithm may not be in accordance with the facts but its results may satisfy the functional requirements of the structure. Every code writer and code evaluator should be aware of this antithetical position.

When the time came for the committee to make design recommendations, they started with “characteristics” unanimously recognized as correct. The first two are worth repeating.

1. *Both the web reinforcement and the concrete compression zone contribute to the shear capacity of the member.*

2. *The web reinforcement not only helps carry part of the total shear, but it also increases the ability of the compression zone to resist shear.*
The first “characteristic” suggested that the truss analogy was incorrect. But there was a subtle change from Richart’s vision. The way to “see” the shear resistance was through the mechanism illustrated in Fig. 9. With the section cut as it was, it made sense if the possibility of friction along the inclined crack (so-called aggregate interlock) and doweling effect of the longitudinal bars were ignored. However, it did not quite make as good sense if one decided to cut the section as shown in Fig. 10. That section too would have to be in equilibrium. But it was difficult to invoke the effect of the web reinforcement on the vertical section unless one accepted that the shear strength of the concrete changed with the section.
The second “characteristic” was interesting. In a beam with web reinforcement, the contribution of the concrete could improve vis a vis that in a beam without web reinforcement. Accordingly, the diagonal-tension cracking load ceased to be waypoint in determining the total shear capacity.

After agreeing that the web reinforcement would yield before shear failure (in members of normal proportions), the joint committee argued their way to the de facto design expression

\[ v_u = K\rho_w f_{wy} + 2\sqrt{f'_c} \]  

\[ v_u \] = yield stress of the web reinforcement

It is interesting that the last term in Eq. 12 appears to be based on the diagonal-tension cracking load despite the “characteristic” that in a beam with web reinforcement the contribution of the concrete can be more than that in a comparable beam without web reinforcement.

The sequence discussed above with respect to shear strength of reinforced concrete beams with web reinforcement was presented primarily to trace the flow of thinking in a case where the profession was faced with problem that did not have a theoretical solution that could match the observations. The theory was not adequate. The observations were limited. The result was an agreement, not a theory, that could be used to design within the limits of a domain limited to the ranges of the variables included in the experiments. Unfortunately, no limits were initially expressed on variables not included in the tests.

Almost half a century after the paper by Anderson (Anderson, 1957), we are still in the state he described. We have a plethora of test results and many theoretical approaches but it is difficult to claim that the problem has been solved. But the code goes on.

**The Load Test**

Perhaps the best example of the tension between the Faustian and the Magian in engineering practice is the load test as defined in Chapter 20 of the ACI Building Code (ACI 318-05, 2005). We shall consider it briefly.

A load test is required if there is doubt (expressed by a licensed engineer) that a part or all of a structure meets the requirements of ACI318-05. Could this doubt be dispelled by analysis? Yes, but in almost a majority of cases involving doubt, the load test is preferred because it tends to be more efficacious in putting an end to dispute.
The magnitude of the test load is defined as

\[ W = 0.85(1.4D + 1.7L) \]  \hspace{1cm} (13)

\( W = \) total test load \\
\( D = \) permanent load \\
\( L = \) live load

The first question that presents itself is why the combination of permanent and live loads within the parentheses does not match any one of the design load combinations included in Chapter 10 of the ACI Building Code. The direct answer is very simple. As it is defined in Eq. 13, the load test has been serving the profession well. Even if the design load has changed, the test load need not change. This is an elegantly pragmatic action but it creates a disturbing disconnect between the design load and the test load.

The combination was once \( 1.4D+1.7L \) before the load factors were subjected to fine tuning, again in the interest of a pragmatic cause: to normalize the load factors for different materials. Well then, why the factor 0.85? Answer: To reduce the likelihood of developing permanent deformations during the test. That seems like a non sequitur. Fundamentally, the load test is to dispel doubts about the strength of the structure with respect to the requirements of the code. Is that question addressed fully if the test load is less than the specified design load, even if we forget about the material (understrength) factors? Answer: No structure load-tested has ever given problems under service. It is hard to deny the fact that the answer is Magian. It is magic.

That oft-repeated response about no structure load-tested ever having given problems embodies in a nutshell the essence of the engineering approach. Once that is accepted as sufficient proof, all other questions about appropriate selection of load level, loaded elements (with respect to number and position) and consequences of cracks observed recede into the background. The load test is not a logical procedure to settle a question of safety but it works. Why? Because it works. As opposed to “theory” cited by those who shaped the design code for the flat slab, the load test is pure engineering. It is a complex and certainly unabashed combination of the Faustian and the Magian.

**Concluding Remarks**

What Oliver Wendell Holmes wrote in 1881 about Common Law is not irrelevant to building codes:

*The life of the law has not been logic: it has been experience. The felt necessities of the time, the prevalent moral and political theories, intuitions of public policy, avowed or unconscious, even the prejudices which judges share with their fellow-men, have had a good deal more to do than the*
syllogism in determining the rules by which men should be governed. The law embodies the story of a nation’s development through many centuries, and it cannot be dealt with as if it contained only the axioms and corollaries of a book of mathematics. In order to know what it is, we must know what it has been, and what it tends to become. We must alternately consult history and existing theories of legislation. But the most difficult labor will be to understand the combination of the two into new products at every stage. The substance of the law at any given time pretty nearly corresponds, so far as it goes, with what is then understood to be convenient; but its form and machinery, and the degree to which it is able to work out desired results, depend very much upon its past.

There is no absolute truth that governs all segments of a building code uniformly. A building code is doomed to vacillations between the Faustian and the Magian. There is no prescription for developing a building code that can be guaranteed to be successful, but there are simple rules, derived from experience, that deserve mention.

1. Keep it simple.

Simple is easier said than done. There needs to be at least one and maybe more members of the code committee who understands what will be understood by the rank and file and who is at once modest (to speak in simple terms) and assertive (to convince the others). It is good to remember that all well understood aspects of structural design are simple and ought to be stated in simple and direct terms. And there is little to be gained by complicated explanations of those aspects that are not well understood. Statements of simple limits will do.

2. Make certain that the users understand the code leads to a structure with minimum safety standards.

It should be clear to the engineering community that good engineering does not mean “beating the code” (producing structures that barely satisfy the code). The design community should understand that algorithms included in the code are not necessarily predictive. Their function is simply to help produce a safe and serviceable structure and not to determine response. (In this respect, it is instructive to refer to the title of Table 9.5 (b) of ACI 318-02, “Maximum Permissible Computed Deflections.” Why was it titled “Computed Deflections”?)

3. Stick to a single perspective.

This simple goal is almost unattainable. It is difficult to filter out approaches based not only on safety and serviceability but also on sales of certain products and desires of certain individuals to push pet procedures. Still, it is worth the try.

Methods and procedures may be included in a code if they are governed by judgment. For example, the definition of how reinforcement should be distributed in a flat plate can indeed be accomplished by analysis but that option can lead to wide and awkward variations in practice. Certainly, minima and maxima for sizes and amounts of reinforcement belong in the code even though some of them can be determined theoretically.

5. Do not change the code unless dangerous practices are perceived or failures occur in the building inventory and do not change the code unless the new clauses/procedures are checked by not less than three practicing engineers who have not been involved in their development.

Above all, do not change the code because of tests in the laboratory or theory unless their results are confirmed in the field.

6. Do not fine-tune the code from version to version.

Even if the changes result in sensible improvements, it is preferable to stay with the original requirements unless the change makes a difference of more than 20%.

7. Do not change the location of code clauses in the interest of logical flow.

Reshuffling clauses in the code or redefining existing notation are likely to lead to omissions and errors in design.

8. Do not revise the code in less than seven years.

Indeed, if the code has to be changed every few years, it may be claimed that it is continually immature and should not have been set out as a code in the first place. Advances in knowledge do not represent good reasons for changing the code. Errors in practice do. Every time the code clauses are revised or new ones proposed, they need to be checked in several design offices by professionals who have had nothing to do with their development.

9. A building code has to appeal to the local building technology and to local conditions.

It is wrong to apply a code vetted in a northern climate to buildings in a southern climate, to base the code in a seismic region on experience in a seismically inert region, or to adopt in a cold climate the fire-related clauses of a code developed in the tropics. Physics may be universal but engineering practice is not.
10. Keep the code committee small.

More than twelve imams diffuse the responsibility. The larger a committee is, the more likely it is to produce nonsense for which none feels responsible. Above all, prefer the wise to the brilliant in the selection process.

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