CURE | Code Updates for Reduction of Embodied-Carbon

PHASE 1: FINAL REPORT

July 8, 2026

Abbie Liel (University of Colorado), Ian McFarlane (Magnusson Klemencic Associates), Ross Corotis (University of Colorado), Michael Gryniuk (CORA Structural), Emily Guglielmo (Martin/Martin), Jim Harris (J.R. Harris & Co), Shana Kelley (KPFF Consulting Engineers), Dirk Kestner (Walter P. Moore), Ray Pugliesi (Degenkolb), Greg Soules (CB&I)

This project was funded by a grant from the Charles Pankow Foundation to the University of Colorado.

Funding to the Charles Pankow Foundation was from the Magnusson Klemencic Associates (MKA) Foundation, the American Society of Civil Engineers (ASCE) Structural Engineering Institute (SEI), the National Council of Structural Engineers Association (NCSEA) Foundation, the American Concrete Institute (ACI) Foundation, and the American Institute of Steel Construction (AISC).

NCSEA Foundation Contributors (x26) included: DCI Engineers, Gilsanz Murray Steficek, McNamara Salvia SE, Hatfield Group, Saiful Bouquet, Severud Associates, Simpson Gumpertz & Heger, Thornton Tomasetti, Uzun+Case, O'Donnell & Naccarato, OLMM Consulting Engineers, Linchpin Structural Engineering, JQOL Global LLC, SSOE, True North Structural Engineering, SMA Architecture + Design, KPFF Consulting Engineers, Gardner Engineering & Consulting PLLC, Paradigm Structural Engineers, Equilibrium Engineers LLC, FTF Engineering, Zeltmacher Structural Design Engineering, Buehler, Degenkolb Engineers, and Froelich Engineers.

Acknowledgments

We are grateful to our two industry champions, Ron Klemencic (MKA) and Jerome Hajjar (Northeastern University), and Charles Pankow Foundation Executive Director, Stuart Harrison, for their advice and encouragement. Representatives from SEI, NCSEA, ACI Foundation and AISC served on our Advisory Panel. We also benefited from collaboration with our global partners who are addressing Eurocodes and the Chinese Building codes, including Will Hawkins (University of Bath), Daniel Safarik (Council on Vertical Urbanism), and their teams.

Table of Contents

INTRODUCTION

Reduction in the emission of greenhouse gases is essential and urgent for mitigation of global climate change. The built environment is a major end user for many high impact materials that are associated with significant embodied carbon. As a result, embodied carbon attributed to the construction of buildings and infrastructure is a primary contributor to global greenhouse gas emissions, accounting for nearly 15% of total emissions[1].

Clear, targeted action is required if substantial reductions in embodied carbon are to be achieved. This report identifies and prioritizes provisions in current building codes and standards which, if appropriately modified, could result in a substantial reduction in embodied carbon in new buildings. While there are many strategies in design and construction that can reduce the embodied carbon impacts, building codes and standards provide a barrier that may limit some of the potential reductions due to prescriptive or conservative code requirements. However, there is also an opportunity here, because prescriptive provisions in building codes and standards are used in the vast majority of designs for new buildings. Therefore, a reduction in conservatism could have both immediate and broad impact.

This project reexamines these codes and standards through the lens of embodied carbon, representing a systematic rethinking of some of the fundamental engineering design requirements to identify the potential for embodied carbon reduction. The focus of this effort is on ASCE 7 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2022), *ACI 318 Building Code Requirements for Structural Concrete* (ACI, 2025), and *AISC 360 Specification for Structural Steel Buildings* (AISC, 2022). These documents are the basis for structural requirements in the International Building Code (ICC, 2024), which is the model building code adopted by most jurisdictions in the U.S., as well as much of the world.

The goal of this effort is to identify code changes that will result in embodied carbon reductions without compromising the necessary safety, durability, or serviceability of buildings. Many of these changes will be controversial and require fundamental changes in approaches that have been historically used in building codes, or require reconsideration of prescriptive requirements that have governed for decades. Given the risks of climate change and need for impact, broad and fundamental changes should be considered.

---

[1] Architecture 2030 using data from IEA & Statista: https://www.architecture2030.org/why-the-built-environment/why-buildings/

PRIORITY CODE CHANGES

This report describes 11 priority code changes (PCC) that could be made to building standards to reduce embodied carbon. These priority code changes include recommendations for consideration for ASCE 7 (ASCE, 2022), ACI 318 (ACI, 2025), and AISC 360 (AISC, 2022). The provision changes could reduce embodied carbon by reducing the total quantity of materials (e.g., reducing the required section sizes), reducing the carbon intensity of materials (e.g., enabling use of lower strength concrete), or both. The report details why each of these code changes is a priority, outlines and illustrates the potential change, its potential impacts on embodied carbon, and describes next steps and potential roadblocks. Some of these proposals could reduce embodied carbon of the structure by as much as 12%; even reductions on the order of 1-3% are meaningful given the quantity and scale of the building industry and its contribution to embodied carbon.

PRIORITY CODE CHANGE NUMBER	PRIORITY CODE CHANGE	SUMMARY
PCC1	Re-evaluate live loads	Reduce design live loads for some occupancies by collecting new data that better defines current building use and reducing conservatism.
PCC2	Apply reliability framework for gravity loads	Develop a reliability-based framework for live loads in line with reliability-based development of environmental loads, reducing conservatism inherent in the current maximum approach with a 1.6 load factor.
PCC3	Develop reliability framework for serviceability assessment	Develop a reliability-based framework for serviceability assessment in line with reliability-based development for ultimate limit states, reducing conservatism and variability.
PCC4	Redefine gravity deflection limits based on building use	Define deflections limits under gravity loads that account for differences in occupancy, enabling serviceability design that meets performance and reliability targets.
PCC5	Reexamine long-term deflection multipliers	Examine long-term deflection multipliers to ensure compatibility with current materials and building practices, and avoid unnecessary conservatism.
PCC6	Improve accounting for steel overstrength	Account more directly for expected steel strength in design, reducing unnecessary material use in strength-controlled components.
PCC7	Improve fireproofing requirements	Reduce fireproofing material usage by enhancing use of performance-based procedures, and reducing conservatism in existing prescriptive procedures.
PCC8	Revamp prescriptive concrete mix proportioning	Reduce cement content of elements controlled by durability requirements for water-cement ratio and minimum strength.
PCC9	Revamp concrete durability requirements	Reduce cement insensitivity of concrete by reducing conservatisms in concrete durability requirements.

PCC10	Refine Effect of Concrete Slab Strength on Column Axial Force Transmission	Enable use of later-age compressive strengths in evaluating slab-column force transfer and bearing stresses.
PCC11	Reduce early strength requirements for post-tensions slabs	Reduce required initial compressive strength for post-tensioning anchorage installation.

The full list of code provisions considered is provided in the Appendix.

METHODOLOGY USED TO DEVELOP PRIORITIES

Overview

Our approach to developing priority code changes to reduce embodied carbon relied on informed judgment of a committee of experts. These experts were selected due to their expertise in structural design, embodied carbon, and in development of codes and standards, and include practicing engineers and academics. The committee met monthly to refine, share, and collaborate on ideas.

Brainstorming Provisions to Reduce Embodied Carbon

We first embarked on a broad brainstorming effort to identify as many candidate provisions in codes and standards that could reduce embodied carbon as possible. To do so, we assigned each member of our team to one or more of the key codes and standards documents. These individuals identified items in those documents through careful review of the contents and provisions and through conversations with colleagues who are active in embodied carbon efforts, in the codes and standards space, and/or experienced designers. Additionally, our team solicited input from the industry advisory panel, our financial supporters, and non-involved practicing engineers. We focused on identifying provisions that could potentially be modified without compromising other structural performance goals.

We compiled these lists to create master list of 76 potential provisions in codes and standards which, if modified, could lead to reductions in embodied carbon, including 39 related to ASCE 7, 13 related to AISC Structural Steel Specifications, 18 related to ACI 318, and 6 others. Each code provision was associated with a brief description and a statement of potential impact.

Evaluating Impact and Feasibility of Provisions

We ranked the candidate provisions according to multiple measures of embodied carbon impact and feasibility. Each member of the committee was asked to rank each potential code provision according to the broad impact, specific impact, and feasibility criteria.

Broad impact

Under broad impact, we considered: "What is the broad impact of this change proposal on the industry as a whole? Does it impact a carbon reduction on a broad range of buildings?" Key metrics for consideration here were the number and classes of buildings potentially affected. Broad impact was evaluated on a scale from 0 to 5, where 5 indicated significant impact on embodied carbon, and 0 indicated no impact.

Specific impact

Under specific impact, we considered: "What is the specific impact of this code provision? Will it permit a significant embodied carbon reduction where this provision applies?" Here, we considered changes that could have very large impact on a narrower subset of buildings. Specific impact was evaluated on a scale from 0 to 5, where 5 indicated significant impact on embodied carbon, and 0 indicated no impact.

Feasibility

Under feasibility, we considered: "What is the feasibility of the proposed change?" Aspects related to feasibility included an assessment of the research effort that is needed, and whether codes and standards committees were likely to be receptive to the change. Feasibility was evaluated on a scale from 1 to 5, where 5 indicated that not much additional research was needed and code committees were likely to be receptive to the change. A rating of 1 indicated significant research and fundamentally new material would be needed.

Other considerations

Other co-benefits of changing code provisions were considered, but not quantitatively ranked. These included considerations such as cost, health, and equity.

Identifying Priority Provisions

To prioritize the provisions, we compiled the committee members' rankings for broad impact to identify those provisions that were perceived to have the most significant impact on embodied carbon reduction. We then examined provisions with high specific impacts that we may have missed based on this ranking strategy, and prioritized more feasible provisions higher. We also combined some of the ideas which could be efficiently considered together. This process was informed by the quantitative rankings and modified through discussion of other considerations.

Semi-Quantitative Evaluation of Embodied Carbon Impact

To further evaluate the potential impact of the potential code change proposals, we conducted a semi-quantitative evaluation of the embodied carbon impact. This evaluation included a high-level life-cycle assessment for representative archetype buildings. The archetypes included low (5 story), mid (10 story), and high-rise (20 story) buildings of both concrete and structural steel construction variants. The studies were limited only to the impact of the structural system, including foundations.

The scope of the life cycle assessment was limited to the product phase, including raw material supply, transport, and manufacturing impacts. The studies evaluated a baseline assumption as the current building code, and then compared these results with a range of impacts estimated by the committee for each potential code change proposal. While this is not a definitive and complete embodied carbon assessment, it is intended to provide a relatively consistent semi-quantitative evaluation of each potential code change proposal. The results of this evaluation are presented with each potential code change proposal.

PCC1: RE-EVALUATE LIVE LOADS

Introduction

Structural design codes define design live load values intended to represent the maximum expected loading conditions for different building occupancies. These values have historically been derived from a combination of load surveys, theoretical modeling, and engineering judgment. However, many of the foundational studies informing these provisions were conducted a half-century ago. As a result, the live load values currently specified in design codes for some of the most common occupancies likely do not reflect today's (current) building usage patterns or updated statistical understanding of occupancy loads.

Live loads play a critical role in structural design because they influence member sizing, system selection, and overall structural material quantities. In many buildings, particularly offices, residential buildings, and parking garages, gravity load combinations involving live load govern structural member design. As a result, overly conservative live load provisions may resul in an increase structural material usage and associated embodied carbon.

Current Code Provisions

Current building codes and standards (e.g., ASCE 7 Chapter 4 and Table 4.3-1) specify uniformly distributed and concentrated design live loads for various occupancies. For example:

- Office floor live load: 50 psf
- Parking garage live load: 40 psf

These design live load values were developed using data and assumptions that, in many cases, date back several decades. The values for several of the most common occupancies are based upon well-vetted surveys that were state-of-the-art at the time, but for some occupancies (particularly offices), the usage of spaces has changed with time (e.g., reducing paper storage), and there is a potential for better survey techniques today, including use of modern sensing and datasets (e.g., data on occupancy duration, tenant buildout specifications, or use of existing cameras to extract loads). For occupancies for which surveys were not available, the live loads relied on limited measurements of occupancy loads and simplified assumptions about maximum loading conditions.

These design live load values are incorporated into structural design load combinations. For gravity load combinations, a factor of 1.6 is typically applied to the live load, meaning that structural elements are designed for significantly higher loads than the design live load specified in the table.

For example:

- Office design live load: 1.6x50psf = 80psf
- Parking garage design live load: 1.6x40psf = 64psf

These load factors are intended to account for uncertainty in load magnitude and occurrence, and are tied to reliability-based calibration of structural design provisions. These aspects are discussed in more detail in PCC2.

Potential Change

A potential code change would involve reevaluating the design live load values for certain occupancies using up-to-date measurements of building usage to develop statistical data and advances in probabilistic modeling.

New studies could examine:

- Measured occupancy loads in today's buildings.
- Updated statistical distributions of maximum loads.
- Differences in today's building use patterns compared to historical data.
- Probabilistic models for load occurrence over the life of a structure.

One possible approach is to recalibrate live load provisions based on actual probability distributions of loads. This approach could also provide values of sustained or transient live loads, which would be lower than the uniform live load value. Another potential approach would involve performance-based load determination, where designers could demonstrate acceptable reliability using project-specific load assumptions and statistical models.

These approaches would aim to provide a more accurate, and likely less conservative, representation of real-world conditions while maintaining the target reliability/level of safety.

Technical Background and Literature

The technical basis for live load provisions historically comes from occupancy load surveys and probabilistic models of maximum load occurrence over time. For example, office live load values were originally developed using survey data and theoretical models intended to estimate the maximum load expected during a 50-year building life. The design live load value was set near the mean of the distribution, with load factors applied in design combinations to account for uncertainty and achieve the desired reliability.

Illustrative examples highlight the conservatism inherent in current provisions.

- Measurements of real-world crowd loading conditions show that typical office occupancies often produce loads well below the design value. In one illustrative example, an office gathering produced an equivalent distributed load of approximately 29 psf, while a densely packed office meeting produced approximately 48 psf - both below the code design value of 50 psf before load factors applied. Similarly, large crowd events can produce loads on the order of 60 psf, however, a typical assembly live load value in the code requires 100 psf, which is unrealistic. See Corotis et al. (2023).

- Up-to-date assessments of parking garage loads show that most parking garage loads are lower than the design value. The current code provision of 40 psf, when combined with the 1.6 load factor, results in a design load of 64 psf. Notably, even in a hypothetical scenario where the heaviest commercially available vehicles in 2025 that are sized to fit conventional parking garages parked unrealistically tightly together, the resulting equivalent distributed load is approximately 64 psf, corresponding to the current strength design load. See Corotis et al. (2025).

Key References

- Corotis, R., Hooper, J., & Klemencic, R. (2023). Design live loads for office gathering spaces. *Journal of Structural Engineering*, 149(12).
- Corotis, R., Arwade, S., & Ha, K. D. (2025). *Safe and sustainable parking garage live loads in the age of the electric vehicle* (RGA #05-24, Final Report). Charles Pankow Foundation.
- Tsay, W.-Y., & Corotis, R. (1983). Probabilistic load duration model for live loads. *Journal of Structural Engineering,* 109(4).

Potential Impact of Change

Re-evaluating live load provisions could have several potential impacts on structural design and construction.

Material Efficiency and Embodied Carbon

Gravity load provisions control the design of the majority of structural members and, in many cases, the load combinations with live loads are critical. If live load values were recalibrated to more accurately reflect current loading conditions, structural members could potentially be designed with reduced material quantities. Even modest reductions in required structural capacity can lead to reductions in:

- Steel tonnage
- Concrete and timber volumes
- Foundation sizes, excavation, and formwork

Because live loads affect a large portion of the structural system, from deck to beams to vertical components to foundations, these changes could have a meaningful impact on material efficiency across many building types.

Alignment With Performance-Based Design

Today's structural engineering practice increasingly relies on performance-based approaches, such as is the case with seismic design. Similar concepts could be applied to live loads by allowing designers to demonstrate acceptable reliability using probabilistic load models.

Improved Consistency with Current Building Usage

Building occupancy patterns have evolved over time. For example:

- Office densities have changed.
- Furniture and equipment loads have shifted.
- Parking garages now accommodate electric vehicles with different weight distributions.

Updated live load provisions using current data could improve alignment between code assumptions and actual building usage.

Semi-Quantitative Embodied Carbon Impact

Due to the wide range of live loads addressed in the building code that could be impacted by this code change, and based on the variability in outcomes that are anticipated, the potential embodied carbon reduction in archetype studies ranges from 2% to 5% for the overall structure. The lower range of impact is likely in low-rise buildings, with high-rise buildings on the higher end of this range.

Next Steps and Potential Roadblocks

Although the potential benefits of revisiting live load provisions are notable, several challenges would need to be addressed.

Potential Next Steps

- Large-scale studies measuring real occupancy loads across building types would likely be required to support code changes, and these data are not currently available. Fortunately, new sensing technologies and visual and other datasets available across large number of buildings present opportunities.
- Any proposed changes would need to demonstrate that revised load provisions maintain equivalent reliability levels relative to existing structural safety targets. This would likely require reliability-based calibration studies.

Potential Roadblocks

- Structural design codes are often seen as conservative and slow to evolve. Code committees typically require strong technical justification and broad consensus before modifying load provisions. However, experience has shown that committees are open to revisiting load values when supported by convincing data and analytical studies. Engineers, building officials, insurers, and owners may initially be cautious about reducing live load values. Careful communication and demonstration projects may be needed to build confidence in updated provisions.

Summary

Certain live load values could be re-evaluated and potentially reduced using current data on building usage patterns, probabilistic modeling, and performance-based approaches. Reassessing these provisions may offer an opportunity to maintain safety and serviceability while reducing unnecessary conservatism and improving material efficiency in structural design.

PCC2: APPLY RELIABILITY FRAMEWORK FOR GRAVITY LOADS

Introduction

The reliability targets and load resistance factor design formulation for gravity loads in ASCE 7 are based on calibration exercises carried out nearly 50 years ago. The primary basis was the dead plus live load combination for two common occupancies: office and residential. Common ratios of live to dead load effects were used for various structural systems. The rationale behind that calibration was that there was general satisfaction with the performance of buildings designed for those particular loads and load ratios. Office and residential occupancy were selected because there were sound data and theoretical models for those two live loads. Even though the common load ratios were different for concrete, steel, and wood, the reliabilities at the ratios common for each system were relatively consistent except at extreme values. Those same calibration exercises were used to inform determination of design wind loads. Over the ensuing decades, there have been many changes in wind loads, and the current ASCE 7 wind load provisions deliver reliabilities that are much closer to the targets in Chapter 1 of ASCE 7. However, the reliability basis for dead and live loads has not been reevaluated for decades.

Current Code Provisions

Code design live loads today are based on the maximum load expected in 50 years, with a load factor larger than 1.0 to those loads for the purpose of checking strength limit states. The load factor for live loads (1.6) that was adopted from the aforementioned calibration studies was applied to all live loads. The 50-year reference period was often described as the normal service life for a building.

For example:

- Office design live load: 1.6x50psf = 80psf
- Ice rink live load, based on two Zamboni-type ice resurfacing machines: 1.6x250psf = 400psf

Figure 1. (a) An office live load of 48 psf; (b) 1 Zamboni-type ice surfacing machine.

Potential Change

Compared to Topic PCC1, this topic is focused on a broader picture of live loads, potential efficiencies and performance benefits to be realized by revising the format of how we specify gravity loads and their combinations.

This revision should reexamine the load factor of 1.6, both in terms of its implied risk and its variance for different types of occupancies. For instance, it is not obvious that 1.6 times the design load for office occupancies has the same implied risk as 1.6 times the design load for ice rinks, assembly areas or storage facilities. Variable load factors for different live loads could be implemented by using a generic symbol for the factor on live load in the load combinations and then specifying the appropriate load factor for each live load in the table of live loads (in practice, two factors for each, one as the leading transient load and another for the arbitrary point in time value). It would also be appropriate to evaluate the live load reduction factor and whether it should be occupancy dependent. It is envisioned that live loads could also be defined with separate sustained and transient loads.

In addition, the current load factors on gravity loads ignore the differing reliability targets that we have for different Risk Categories of structures, whereas ASCE 7 directly specifies differences in environmental loads for different risk categories. Past studies have shown inconsistency of reliability levels implied by various loads. Seismic and wind provisions have examined these in terms of recent performance-based design updates, but no such examination has been made of gravity loads.

Technical Background and Literature

Although the fundamentals set forth in the report that led to our current formulation of live loads (NBS 577) are still valid, there have been several advances. The introduction of reliability targets that vary with Risk Category in ASCE 7 has not affected the way dead and live loads are specified, although it has affected seismic, wind, and snow loads. The ISO standard for structural reliability (ISO 2394) was updated in 2015 to incorporate risk-informed procedures.

Key References

- Corotis, R. (1985). Probability-based design codes. *Concrete International*, 7(4).
- Culver, C. G. (1976). *Survey results for fire loads and live loads in office buildings* (National Bureau of Standards Special Publication 85). National Bureau of Standards.
- Ellingwood, B., Galambos, T., MacGregor, J., & Cornell, C. A. (1980). *Development of a probability based load criterion for American National Standard A58* (National Bureau of Standards Special Publication 577). National Bureau of Standards.
- Hasofer, A. M. (1968). Statistical model for live floor load. *Journal of the Structural Division*, 94(ST10).
- ISO (2015). General principles on reliability of structures. 4th Edition.

- Liel, A. B., DeBock, D. J., Harris, J. R., Ellingwood, B. R., & Torrents, J. M. (2017). Reliability-based design snow loads. II: Reliability assessment and mapping procedures. *Journal of Structural Engineering,* 143(7).
- Luco, N., Ellingwood, B. R., Hamburger, R. O., et al. (2007). Risk-targeted versus current seismic design maps for the conterminous United States. *In SEAOC 2007 Convention Proceedings.* Structural Engineers Association of California.
- McAllister, T. P., Wang, N., & Ellingwood, B. R. (2018). Risk-informed mean recurrence intervals for updated wind maps in ASCE 7-16. *Journal of Structural Engineering,* 144(5).
- Mitchell, G. R., & Woodgate, R. W. (1970). *Floor loadings in office buildings – The results of a survey* (CP3/71). Building Research Station.

Potential Impact of Change

Gravity loads control the design of many components, contributing to a multiplying effect to embodied carbon: floor systems dead and live loads affect the floor system, the vertical components of the gravity system and the foundations. It is suspected that there might be significant conservatism in the current approach. Occupancies ripe for update include offices, residential structures, storage warehouses and parking garages. These are very common engineered structures, and ones for which the gravity loads often control design with live loads being a significant contributor.

Material Efficiency and Embodied Carbon

Live load provisions often control the design of structural elements, particularly in office, residential, and parking structures. With a new framework for determining live loads, there is substantial potential that structures could be designed with reduced material quantities. Even modest reductions in required structural capacity can lead to reductions in:

- Steel tonnage
- Concrete volumes
- Foundation sizes

Alignment With Performance-Based Design

The new studies called for in Topic PCC1 will provide probabilistic information on sustained, transient and lifetime loads. Today's structural engineering increasingly relies on performance-based approaches, particularly in areas such as seismic design. Similar concepts could be applied to gravity loads by allowing designers to demonstrate acceptable reliability using probabilistic load models.

Demonstrated Reliability

The proposed approach could reduce live load actions while demonstrating that reliability targets are met. It would also potentially harmonize treatment of live loads with environments loads.

Semi-Quantitative Embodied Carbon Impact

Due to the wide range of loads addressed in the building code that could be impacted by this code change, and based on the variability in outcomes that are anticipated, the potential embodied carbon reduction in archetype studies ranges from 2% to 10% for the overall structure. The lower range of impact is likely in low-rise buildings, with high-rise buildings on the higher end of this range.

Next Steps and Potential Roadblocks

Although the potential benefits of revisiting live load provisions are notable, several challenges would need to be addressed.

Potential Next Steps

- Improving the reliability-based framework for gravity loads will require interrogating the reliability provided by the current framework to identify modifications needed to provide more consistent reliability and opportunities to reduce conservatism. This will require updated probability models for live loads (which would ideally be supported gathering data related to loading on current uses and construction of buildings, as described in Topic PCC1). A first step could be a "survey" of those load categories expected to dominate future construction. A study will need to consider multiple materials and systems to understand if we can maintain the independence of resistance and load factors.
- To conduct the studies called for above, significant research may be required to evaluate the reliability-implicit considerations.

Potential Roadblocks

- Design of many structural elements is not strength controlled. Other load cases, such as loads carried during construction may need to be considered. There is also the potential that safety expectations or analysis would actually produce an increase in loads, although this outcome should not be considered in a negative light.
- Engineers, building officials, and insurers may initially be cautious and reluctant to change live load values from the expected lifetime maximum to a fuller probabilistic analysis. Prior experience has indicated difficulty in getting code committees to make significant changes to loads, load and resistance factors, and computed reliability levels. Loads and factors on gravity loads have remained constant for a significant period, producing comfort with the current practice and institutional inertia. Careful communication and demonstration projects may be needed to build confidence in updated provisions.

Summary

A reliability-targeted approach to live loads could refine code minimum loads, load factors, live load reduction factors, and treatment of Risk Categories, providing occupancy-specific values. This calibration could reduce unnecessary conservatism and improving material efficiency in structural design.

PCC3: DEVELOP RELIABILITY FRAMEWORK FOR SERVICEABILITY ASSESSMENT

Introduction

Serviceability governs member selection or sizing of many elements. However, load combinations for purposes of gravity serviceability checks and deflection determination vary across materials and standards (AISC, ACI, PTI, ASCE 7), with inconsistent or nonexistent reliability bases. Furthermore, the standard of practice for checking serviceability varies across different regions, firms, and structure types. In many cases, engineers use ASD load combinations intended for strength checks to evaluate serviceability. Many serviceability limits are not tied to quantifiable reliability targets or performance outcomes. Serviceability limits that do have some basis in reliability theory have not been revisited since the 1980s.

Current Code Provisions

The current code provisions for serviceability load combinations are found in the commentary to the non-mandatory ASCE 7-22 Appendix C. This text includes these statements:

“Serviceability limits (e.g., maximum static deformations or accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be checked using appropriate loads for the limit state being considered.”

The commentary to Appendix C further reports serviceability loads for snow and wind loads that have “usually led to acceptable performance in the past”.

Potential Change

Possible changes to these existing provisions and recommendations would be to establish consistent, reliability-informed load combinations for deflection across occupancies and materials. These changes would require development of a framework of objectives with reliability targets that could be different for different occupancies, systems, materials and time frames. This revised framework – comprising updated serviceability limits, serviceability load factors and loads – could then be placed in a new chapter in ASCE 7 or alternative document to establish consistent lower bounds on serviceability.

Technical Background and Literature

Serviceability criteria are highly decentralized, with variations in publications produced by different standards and material organizations. Several of the most used references in industry are listed below. Ellingwood (1989) and Galambos and Ellingwood (1986) describe the historical basis of the treatment of serviceability.

Key References

- American Concrete Institute. (2020). *Report on deflection of nonprestressed concrete structures* (ACI 435R-20).
- West, M., Fisher, J., and Griffis, L. (2004). Serviceability design considerations for steel buildings. American Institute of Steel Construction (Steel Design Guide No. 3, 2nd ed.).
- American Society of Civil Engineers. (2022). *Minimum design loads and associated criteria for buildings and other structures* (ASCE 7-22), Commentary Appendix CC.2.
- Ellingwood, B. (1989). Serviceability guidelines for steel structures. *AISC Engineering Journal*, 26(1).
- Galambos, T. V., & Ellingwood, B. (1986). Serviceability limit states: Deflection. *Journal of Structural Engineering,* 112(1).
- Swensson, K. D. (2012). *Guide to the design of building systems for serviceability: In accordance with the 2012 IBC and ASCE/SEI 7-10*. International Code Council; National Council of Structural Engineers Associations.

Potential Impact of Change

Material Efficiency and Embodied Carbon

Serviceability often governs slab thickness, beam depth, and column sizes—major drivers of embodied carbon. Inconsistent or inappropriate load combinations can lead to unnecessary material use, especially in concrete and post-tensioned systems. Improved clarity on load combinations, combined with improved characterization of long-term behavior (see PCC5), could allow the use of optimized sections with lower carbon.

Alignment with Reliability-Basis and Performance Expectations

Reliability-based deflection criteria would align structural member size with actual performance expectations rather than legacy numbers. Clear, evidence-based limits reduce overdesign while maintaining occupant comfort and building functionality.

Semi-Quantitative Embodied Carbon Impact

Due to the wide range structures and members that could be impacted by revised serviceability criteria, and because some of these members are often designed to meet more stringent serviceability criteria than code minimum, the potential embodied carbon reduction in archetype studies ranges from 1% to 5% for the overall structure.

Next Steps and Potential Roadblocks

Research needs span measurement, modeling/performance assessment, and material properties to characterize long-term performance, requiring multi-year effort.

Potential Next Steps

- There is a need to establish consistent, reliability-informed load combinations for deflection across occupancies and materials. This task requires development of a reliability framework and exercising that assessment of the reliability across different systems, materials and time frames. It is also necessary evaluate the reliability implications of currently used serviceability load combinations and identify inconsistencies between materials and design standards.

- A key step is to gather available field-performance data and published research to better correlate serviceability limits with observed building performance and occupant expectations. However, real-world deflection data is sparse.

Potential Roadblocks

There is a lack of unified ownership, as serviceability provisions live in multiple standards (IBC, ASCE 7, ACI, AISC, PTI). The standard of practice varies across firms, across regions, and across structure types (buildings vs. nonbuildings) making consensus difficult. Code officials and designers may resist changes due to perceived risk or liability. In addition, a move to codify serviceability more explicitly might be seen as counter to the performance-based design direction of the profession. This will require careful coordination across multiple standards and industry groups.

Summary

Reliability calibration focused on the serviceability limit state could provide more robust recommendations on load combinations and deflection limits that enables serviceability performance, without overdesign that amplifies embodied carbon.

PCC4: REDEFINE GRAVITY DEFLECTION LIMITS BASED ON BUILDING USE

Introduction

Deflection limits are long-standing prescriptive criteria aimed at preventing cracking, ponding, façade damage, and occupant discomfort. The standard of practice for checking serviceability varies across different regions, firms, and structure types. Many serviceability limits are not tied to quantifiable reliability targets or performance outcomes. Serviceability limits that do have some basis in reliability theory have not been revisited since the 1980s.

Current Code Provisions

The current code provisions for gravity deflection limits are found in the commentary to the non-mandatory ASCE 7-22 Appendix C and IBC Section 1604.3. IBC Table 1604.3 provides limits for live loads, snow loads, wind loads, and dead plus live loads for roof members, floor members, exterior walls, interior partitions, farm buildings and greenhouses.

For example:

- Under live loads, the deflection of floor members should not exceed l/360.
- Under live loads, the deflection of roof members not supporting ceilings should not exceed l/180.

Potential Change

Possible changes to these existing provisions and recommendations would be to reexamine the deflection limit values and justify these limits based on building use or function rather than blanket, generally applicable, values. These revised limits would then be placed in a new chapter in ASCE 7, or integrated within the current IBC table. These limits could be evaluated to show that they maintain reliability targets (See Topic PCC3).

Technical Background and Literature

Early deflection limits were often rules-of-thumb of allowable spans based on beam depth. These rules were generally set to prevent visually obvious sagging and to protect plaster ceilings from cracking. Some of these rules have remained essentially unchanged for more than a century (Ellingwood 1989), despite changes in building practices including installation of nonstructural components. As such, many of the published sources that contain deflection limitations used today date back to these early rules-of-thumb. Several of the most used references in industry are listed below.

Key References

- American Concrete Institute. (2020). *Report on deflection of nonprestressed concrete structures* (ACI 435R-20).
- West, M., Fisher, J., and Griffis, L. (2004). Serviceability design considerations for steel buildings. American Institute of Steel Construction (Steel Design Guide No. 3, 2nd ed.).
- American Society of Civil Engineers. (2022). *Minimum design loads and associated criteria for buildings and other structures* (ASCE 7-22), Commentary Appendix CC.2.
- Ellingwood, B. (1989). Serviceability guidelines for steel structures. *AISC Engineering Journal,* 26(1).
- Galambos, T. V. (1973). *Structural deflections: A literature and state-of-the-art survey* (Building Science Series Vol. 47). US National Bureau of Standards.
- Galambos, T. V., & Ellingwood, B. (1986). Serviceability limit states: Deflection. *Journal of Structural Engineering,* 112(1).

Potential Impact of Change

Material Efficiency and Embodied Carbon

Serviceability often governs slab thickness, beam depth, and column sizes—major drivers of embodied carbon. Robust examination of deflection limits could provide an opportunity to reduce the impacts of serviceability on design where these are not needed to ensure performance

Alignment with Reliability-Basis and Performance Expectations

Reliability-based deflection criteria would align serviceability's impact on design (e.g., in structural member size) with actual performance expectations rather than legacy numbers. Clear, evidence-based limits reduce overdesign while maintaining occupant comfort and building functionality.

Semi-Quantitative Embodied Carbon Impact

Due to the wide range structures and members that could be impacted by revised serviceability criteria, and because some of these members are often designed to meet more stringent serviceability criteria than code minimum, the potential embodied carbon reduction in archetype studies ranges from 1% to 5% for the overall structure.

Next Steps and Potential Roadblocks

The next steps and roadblocks listed for PCC3 are relevant here. Additional considerations for deflection limits are as follows.

Potential Roadblocks

A challenge to development of graduated limits based on building use/function rather than blanket values is the need to define a clear rationale for determination of these limits. This process for evaluating and defining these limits would likely require broad input from structural and nonstructural industry representatives, as well as the collection of data to show the suitability of different limits.

Summary

Current application of deflection limits is highly inconsistent and likely conservative. Redefining deflection limits to be used in serviceability criteria could reduce overdesign where it is not needed to maintain serviceability performance.

PCC5: REEXAMINE LONG-TERM DEFLECTION MULTIPLIERS

Introduction

In reinforced concrete structures, phenomena such as shrinkage and creek cause time-dependent deflections that amplify the elastic deflections that occur when loads are first placed on the structure. Significant research has shown that these deflections are impacted by the magnitude of the sustained load, the amount of compression reinforcement, environmental conditions such as temperature, humidity and curing conditions. However, past research and development on these long-term deflection factors for reinforced concrete and post-tension members is dated when considering today's improved materials. Most publications on this subject are dated from the 1960s to the 1980s. The decisions made in arriving at the specific values used in codes and standards such as ACI 318 and ACI/PTI 320 are generally based on conservative judgement as opposed to meeting reliability targets. Long term deflections also affect design of timber structures.

Current Code Provisions

The current code provisions specifying long-term deflection factors are ACI 318-25 Section 24.2.4 and ACI/PTI 320-25 Section 24.2.4.2. The ACI 318 provisions, for example, define multipliers to estimate long term deflections that depend on the time the load is sustained, compression reinforcement.

Potential Change

Revise long-term deflection multipliers (creep/shrinkage) based on material and design advances (e.g., low-carbon mixes, SCM-heavy concretes, common structural configurations) and reliability calibration.

Technical Background and Literature

Existing long-term deflection multipliers are based on simplified relationships based on experimental research on reinforced concrete components. Most of these studies are now several decades old, and did not consider a probabilistic approach.

Key References

- American Concrete Institute. (1992). *Prediction of creep, shrinkage, and temperature effects in concrete structures* (ACI PRC-209-92(08)).
- American Concrete Institute. (2025). *Building code for structural concrete* (ACI 318-25), Commentary Section R24.2.4.

- American Concrete Institute Committee 435. (1966). Deflections of reinforced concrete flexural members (ACI 435.2R-66). *ACI Journal Proceedings*, 63(6), 637–674. https://doi.org/10.14359/7643
- Branson, D. E. (1965*). Instantaneous and time-dependent deflections on simple and continuous reinforced concrete beams* (HPR Report No. 7, Part 1). Alabama Highway Department, Bureau of Public Roads.
- Branson, D. E. (1970). Discussion of "Proposed revision of ACI 318-63: Building code requirements for reinforced concrete.*" ACI Journal Proceedings*, 67(9).
- Branson, D. E. (1971). Compression steel effect on long-time deflections. *ACI Journal Proceedings,* 68(8).
- Branson, D. E. (1977). Deformation of concrete structures. McGraw-Hill.
- Branson, D. E., Meyers, B. L., & Kripanarayanan, K. M. (1970). Time-dependent deformation of noncomposite and composite prestressed concrete structures. In *Symposium on Concrete Deformation, Highway Research Record 324* (pp. 15–43). Highway Research Board.
- Ghali, A., & Favre, R. (1986). Concrete structures: Stresses and deformations. *Chapman and Hall.*
- Post-Tensioning Institute. (2025). *Post-tensioned structural concrete* (ACI/PTI 320-25), Commentary Section R24.2.4.2.

Potential Impact of Change

Total deflection limits (immediate deflection plus time-dependent deflection) often govern slab thickness, and beam depth—major drivers of embodied carbon. Overly conservative limits can lead to unnecessary material use, especially in concrete and post-tension systems. Reliability-based time-dependent deflection criteria would align structural size with actual performance expectations rather than legacy numbers. Clear, evidence-based limits reduce overdesign while maintaining occupant comfort and building functionality.

Semi-Quantitative Embodied Carbon Impact

The impact of this potential code change varies significantly between structure types and long term deflection performance, depending on whether member proportioning is governed by code minimum requirements or other more restrictive serviceability criteria. Therefore, the potential embodied carbon reduction in archetype studies ranges from 1% to 5% for the overall structure.

Next Steps and Potential Roadblocks

The next steps and roadblocks listed for PCC3 are relevant here. Additional considerations for deflection limits are as follows.

Potential Next Steps

- Significant data collection would be needed, including on measured in-service deflections for typical building systems and review of more recent experimental studies to characterize trends in long-term deflections with new materials, design practice, and other factors.
- A key step is to develop a probabilistic reliability-calibrated approach to developing long-term multipliers. This will require careful examples to show the benefit of probabilistic rather than deterministic thinking.

Summary

Long-term deflection multipliers have not been re-examined in the context of new (lower carbon) materials, current design practices, or through a probabilistic basis. Re-examination could produce a reduction in embodied carbon for serviceability-controlled elements.

PCC6: IMPROVE ACCOUNTING FOR STEEL OVERSTRENGTH

Introduction

Structural steel design in North America is based on specified minimum yield strengths defined by AISC and ASTM material standards. In practice, modern steel production routinely delivers material strengths above these specified minimum values due to improved manufacturing quality control and production consistency.

This behavior is already recognized in portions of current structural design practice. Seismic provisions use expected-strength factors such as R_y and R_t, and LRFD resistance factors were originally calibrated using statistical relationships between actual and specified material strengths.

The primary question is whether modern steel overstrength creates opportunities for improved efficiency, particularly for strength-controlled gravity elements and specialty seismic applications, while maintaining current reliability targets. By enabling modest reductions in steel quantities where strength governs design, these refinements could directly lower embodied carbon associated with structural steel and achieve measurable reductions in embodied carbon in targeted applications.

Current Code Provisions

Current steel design provisions are based on specified minimum yield strengths referenced by AISC and ASTM standards.

Statistical studies of mill test reports consistently show that actual yield strengths exceed specified minimum values. Depending on steel grade and product type, measured yield strengths commonly range from approximately 1.1 to 1.3 times the specified minimum value, depending on steel grade, thickness, and product type. The variability in measured yield strength is relatively small, with coefficients of variation commonly ranging from approximately 0.05 to 0.10.

Current codes address this behavior in two primary ways:

- Seismic provisions explicitly account for expected material strength through R_y and R_t factors to enforce ductile yielding behavior.
- The development of LRFD resistance factors through reliability calibration implicitly incorporates material overstrength.

As a result, current design provisions already include much of the statistical benefit associated with steel overstrength.

Potential Change

Rather than broadly reducing steel design requirements, a more practical approach may involve targeted use of expected-strength concepts in applications where strength directly controls design, allowing the designer to use expected steel strengths larger than the minimum value in selected strength-controlled elements such as short columns, braces, tension members, and gusset plates. Improved coordination between designers, fabricators, and steel producers regarding material-property expectations could inform and reduce conservatism in design based on nonlinear and performance-based seismic analysis. Modern material production data could also be used to reevaluate calibrated LRFD factors.

Technical Background and Literature

Multiple statistical studies have confirmed that modern structural steel materials routinely exceed specified minimum yield strengths while maintaining relatively low variability.

Brockenbrough (2003) evaluated more than 38,000 tensile tests for commonly used structural steel grades and found average yield strengths ranging from approximately 1.1 to 1.3 times specified minimum values, with coefficients of variation generally between 0.05 and 0.10. Earlier LRFD calibration studies by Galambos and Ravindra (1978) incorporated similar statistical relationships into the development of LRFD resistance factors, demonstrating that current provisions already account for some degree of material overstrength.

Additional studies by Liu and Astaneh-Asl (2007) and Liu (2016) confirmed that HSS and rolled steel products consistently exhibit measured strengths above specified minimum values. For HSS seismic design, this material overstrength has direct design implications because relatively high R_y values can increase connection and adjacent-member force demands. ASTM A1085 was developed in part to improve material consistency and reduce yield-strength variability in conventional HSS products. Subsequent experimental studies by Kaldestad (2022) and Bergendahl (2021) showed that this improved material consistency produced limited improvement in brace seismic performance, indicating that reduced material variability alone does not necessarily result in proportional system-level benefits. Even so, ASTM A1085 remains significant from a design-force perspective because it permits reduced R_y and R_t factors for adjacent force-controlled steel elements.

Current HSS seismic R_y and R_t provisions appear reasonably calibrated to modern material data, and existing codes already permit the use of tested material strengths in certain applications where noted in AISC 341-22 Section A3.1 and Commentary F4.

Buckling Restrained Brace (BRB) systems also rely heavily on testing and expected-strength concepts. Although this system would be a prime candidate for coordination of steel strength variation between the fabricator and designer, brace strength is typically primarily adjusted through brace core sizing rather than coordination of project-specific material strengths, highlighting the practical difficulty of implementing expected-strength assumptions across entire projects.

For rolled steel sections and moment-frame systems, additional efficiency gains are likely limited because many systems are governed by drift, stiffness, connection detailing, or serviceability requirements rather than pure strength demands.

Key References

- American Institute of Steel Construction. (2022). *Seismic provisions for structural steel buildings* (ANSI/AISC 341-22).
- Bergendahl, M. (2021). *Experimental investigation to compare the cyclic response of A500 and A1085 HSS braces* (Master's thesis, University of Washington).
- Brockenbrough, R. L. (2003). MTR survey of plate material used in structural fabrication, *AISC Engineering Journal,* 1st Quarter.
- Galambos, T. V. (1978). Load and resistance factor design, *AISC Engineering Journal*, 3rd Quarter.
- Kaldestad, J. (2022). *Experimental investigation into the cyclic response of A1085 HSS braces* (Master's thesis, University of Washington).
- Liu, J. (2016). Updates to expected yield stress and tensile strength ratios for determination of expected member capacity in the 2016 AISC seismic provisions, *AISC Engineering Journal*, 53(4).
- Liu, J., Sabelli, R., Brockenbrough, R. L., & Fraser, T. P. (2007). Expected yield stress and tensile strength ratios for determination of expected member capacity in the 2005 AISC seismic provisions, *AISC Engineering Journal*, 44(1).

Potential Impact of Change

Recognizing a bounded portion of steel overstrength in structural design could have several potential impacts. The embodied-carbon benefit of this change would come from reducing steel tonnage in targeted components where design capacity is governed by yield or tensile strength.

Material Efficiency and Embodied Carbon

Although broad reductions in steel quantities are unlikely, targeted efficiency gains may be achievable for strength-controlled elements.

Potential applications include:

- braces,
- short columns,
- tension members,
- plate elements,
- and gusset plates.

Even modest reductions in steel tonnage may contribute to embodied carbon reduction because gravity systems influence floor framing, vertical load paths, and foundations.

However, many structural systems remain governed by stiffness, drift, vibration, or serviceability requirements rather than strength. Since stiffness depends primarily on modulus of elasticity rather than yield strength, the overall impact on total building material quantities is expected to be incremental rather than transformational.

Seismic and Performance-Based Applications

Expected-strength concepts may provide the greatest benefit in seismic and performance-based design applications where reliable prediction of yielding behavior directly influences the design.

These applications may include:

- nonlinear seismic analysis,
- capacity design,
- buckling restrained brace systems,
- and resilience-based structural design.

Semi-Quantitative Embodied Carbon Impact

The impact will vary depending on the structure type, given that even in predominantly steel structures the embodied carbon associated with concrete (topping, foundations, etc) can be a more significant impact. Therefore, the potential embodied carbon reduction in archetype studies ranges from 0% to 3% for the overall structure.

Next Steps and Potential Roadblocks

Potential Next Steps

A practical next step would be to evaluate whether modern steel production data supports refinement of expected-strength assumptions for targeted applications, particularly material grades commonly used in strength-controlled gravity elements and seismic systems. This could involve:

- updating statistical databases of modern steel material properties,
- evaluating the reliability implications of revised expected-strength assumptions,
- reviewing current ϕ factors to determine whether modern manufacturing consistency supports updated calibration assumptions,
- and identifying structural systems where strength rather than stiffness governs design.

Although the concept has potential benefits, several challenges must be addressed before any code changes could be considered.

Limited benefit from revising existing values

Current LRFD provisions already account for material overstrength through reliability calibration. Any future modification would therefore require careful study to avoid unintentionally double-counting conservatism already embedded within existing resistance factors. For example, existing expected strength factors used in seismic design have already been calibrated using large datasets of mill test reports. Studies supporting the 2016 AISC Seismic Provisions indicate that these factors reasonably represent observed material strength distributions, suggesting that simply reviewing existing R_y values may provide limited additional efficiency gains.

Coordination with steel production

One potential approach would involve using actual mill test data to determine expected yield strengths for specific projects rather than based on statistics of industry wide production. However, this approach presents practical challenges particularly on large projects because steel coupon testing typically occurs after design decisions have already been made. Requiring higher minimum strengths during design could increase the likelihood of rejected material and offset potential efficiency gains. Although this approach may be feasible on small projects, the resulting material savings would likely be limited.

Code adoption challenges

Changes to structural design provisions require strong technical justification and careful reliability calibration. Any proposed modifications would need to demonstrate that the overall reliability of structural systems remains consistent with current safety targets.

Industry acceptance

Design professionals, building officials, and insurers may initially be cautious about incorporating expected material strength into routine design calculations. Additional research and pilot studies may be necessary to build industry confidence.

Summary

The available evidence suggests that the greatest opportunities involve targeted refinement of specialty and strength-controlled applications rather than wholesale modification of routine structural steel design provisions.

PCC7: IMPROVE FIREPROOFING REQUIREMENTS

Introduction

Fire-resistance requirements for structural systems are a fundamental component of modern building codes. These requirements are intended to ensure that structural elements maintain adequate load-carrying capacity during fire exposure, thereby preventing structural collapse and allowing occupants sufficient time to evacuate. In most jurisdictions, fire-resistance ratings are prescribed based on a combination of factors including building occupancy, height, construction type, and the structural system used.

Structural fire protection is typically achieved through the application of passive fire-resistive materials such as spray-applied fire-resistive material (SFRM), intumescent coatings, gypsum board enclosures, or through inherent protection mechanisms such as concrete cover over reinforcement or composite floor systems. In practice, prescriptive code provisions frequently treat broad classes of structural elements similarly—for example, applying uniform requirements to “steel beams” or “steel columns” regardless of member size, redundancy, or thermal capacity. Many of the materials used for fire protection are themselves highly carbon intensive.

While these prescriptive approaches have historically provided a reliable baseline for life safety, they can also introduce conservatism. Some structural systems and structural elements possess significant inherent fire resistance due to their size, redundancy, or other performance characteristics. In these situations, prescriptive fireproofing requirements may result in the application of fire-resistive materials that provide limited incremental safety benefit while increasing construction costs and environmental impacts.

This topic explores potential improvements to current fireproofing provisions that could better align code requirements with actual structural fire performance. The objective is not to reduce safety margins but rather to enable more performance-informed approaches that maintain life safety while avoiding unnecessary material use.

Current Code Provisions

Current building codes determine required fire-resistance ratings primarily through prescriptive tables tied to construction type and occupancy classification. For example, the International Building Code (IBC) and similar model codes specify minimum fire-resistance ratings for structural elements such as:

- Structural frames
- Bearing walls

- Floor assemblies
- Roof assemblies

These ratings typically range from one to three hours depending on building height, occupancy risk, and construction type. This prescriptive approach differs from practice in Europe and Australia where analytical and computational methods are more widely used.

For steel structures, fire-resistance ratings are most commonly achieved through the application of spray-applied fire-resistive material (SFRM) or intumescent coatings. The required thickness of fireproofing is determined through standardized furnace tests that measure the time required for a structural member to reach a critical temperature. Code tables or proprietary listings then provide the thickness of fireproofing required for various member types and shapes.

Although these systems are based on well-established fire testing standards, the prescriptive framework has several limitations:

- Structural members are categorized broadly, often without consideration of member size, thermal mass, or system redundancy.
- Fire scenarios used for testing represent standardized heating curves rather than realistic compartment fire conditions.
- Structural demand during fire events is not typically considered explicitly in prescriptive methods.

As a result, structural elements may receive the same fire protection requirements regardless of their actual structural demand or thermal performance characteristics.

Potential Change

Potential code improvements could follow two complementary paths.

Path 1: Expand Adoption and Acceptance of Performance-Based Structural Fire Design

Existing performance-based design approaches allow engineers to evaluate the behavior of structural systems under realistic fire scenarios. In the United States, ASCE 7 Appendix E provides a framework for performance-based structural fire engineering. However, these methods are not widely used due to limited familiarity among practitioners and inconsistent acceptance by authorities having jurisdiction (AHJs).

Expanding guidance, education, and acceptance of performance-based structural fire design could allow engineers to demonstrate that certain members or systems meet required safety objectives with reduced fireproofing.

Key actions could include:

- Clarifying the use and applicability of existing performance-based provisions in ASCE 7 Appendix E
- Developing design examples and guidance documents
- Promoting coordination between structural engineers, fire protection engineers, and code officials

Path 2: Improve and Validate Prescriptive Requirements

A second approach is to revisit prescriptive fireproofing provisions themselves. Advances in computational modeling, experimental testing, and structural analysis methods make it possible to better evaluate how modern structural systems perform under fire exposure.

Research efforts could evaluate typical structural archetypes and determine whether existing prescriptive tables appropriately reflect current materials, construction practices, and structural demands.

Potential outcomes could include:

- Updated fireproofing thickness requirements for certain structural elements
- Greater differentiation between member sizes and structural configurations
- Recognition of inherent fire resistance provided by composite or massive structural systems

Even modest adjustments to prescriptive provisions could yield meaningful reductions in fireproofing materials when applied across large building inventories.

Technical Background and Literature

Research over the past several decades has demonstrated that structural systems often perform more robustly under fire exposure than prescriptive approaches suggest. Several factors contribute to this performance:

- Structural redundancy within modern framing systems
- Redistribution of loads during heating
- Thermal inertia of large structural members
- Partial fire exposure in realistic compartment fires

Performance-based structural fire engineering has been widely studied internationally, particularly in Europe and Australia, where performance-based fire design is more commonly implemented. Analytical and computational methods allow engineers to model temperature development in structural members and evaluate resulting structural behavior during fire exposure.

Several studies have shown that real structural systems can tolerate higher temperatures or longer heating durations than assumed in standard furnace testing due to load redistribution and system-level behavior.

Despite this research, adoption of performance-based structural fire engineering in the United States remains limited. Barriers include limited familiarity among structural engineers, the need for interdisciplinary collaboration with fire protection engineers, and uncertainty among code officials regarding acceptance of alternative methods.

Continued research and validation are therefore necessary to bridge the gap between academic research and practical design applications.

Key References

- American Society of Civil Engineers. (2020) Performance-based structural fire design: Exemplar designs of four regionally diverse buildings using ASCE 7-16 (Appendix E).
- Bailey, C. G. (1999). The structural behavior of steel columns during a compartment fire in a multi-story braced steel-frame. *Journal of Constructional Steel Research*, 52(2).
- Gernay, T. (2024). Performance-based design for structures in fire: Advances, challenges, and perspectives. *Fire Safety Journal*, 142.
- Liu, J. (2021). AISC steel structures research update: Structural fire engineering. *AISC Engineering Journal*, 58(4).
- National Institute of Standards and Technology. (2010). *Best practice guidelines for structural fire resistance design of concrete and steel buildings* (NIST Technical Note 1681).

Potential Impact of Change

Material Efficiency and Embodied Carbon

Fireproofing materials—particularly spray-applied fire-resistive materials (SFRM)—carry significant embodied carbon impacts associated with raw material extraction, manufacturing, transportation, and field application. Large commercial buildings can require substantial quantities of fireproofing material, particularly in steel-framed construction.

Improved alignment between fireproofing requirements and actual structural fire performance could reduce unnecessary application thicknesses or extent of fireproofing in certain structural configurations. Even modest reductions in fireproofing quantities could result in meaningful reductions in:

- Fireproofing material use
- Associated embodied carbon
- Construction waste and material handling

Because fireproofing is applied broadly across many structural systems, small percentage reductions could produce significant cumulative impacts across the building stock.

Improved Alignment with Structural Performance

Current prescriptive fireproofing provisions often apply similar requirements across broad categories of structural members without fully accounting for:

- Member size and thermal mass
- Structural redundancy
- Composite behavior
- Actual structural demand during fire exposure

More performance-informed provisions could better align fire protection requirements with actual structural behavior under realistic fire scenarios. This could improve consistency between code assumptions and observed system-level fire performance while maintaining equivalent life-safety objectives.

Expanded Use of Performance-Based Design

Greater acceptance and clarification of performance-based structural fire engineering approaches could provide designers with additional flexibility to optimize structural systems and fire protection strategies.

Potential benefits include:

- More efficient integration of architecture, structure, and fire protection systems
- Improved coordination between structural and fire protection engineering disciplines
- Increased use of analytical methods that account for realistic compartment fire behavior and load redistribution

This approach would align structural fire engineering with broader trends toward performance-based structural design already used in seismic and wind engineering.

Constructability and Architectural Benefits

Reducing unnecessary fireproofing thicknesses or extent could also improve project constructability and coordination. Potential benefits may include:

- Reduced application time and labor
- Simplified detailing around complex connections and penetrations
- Reduced conflicts with architectural finishes and MEP systems
- Improved exposed steel architectural opportunities when intumescent coatings or reduced protection approaches are feasible

These benefits would improve overall project efficiency in addition to reducing material usage.

Preliminary Estimated Impact

For the purposes of the CURE study, a reasonable preliminary estimate is that improved fireproofing provisions could reduce fireproofing material quantities by approximately 10–20% in certain structural systems while maintaining equivalent fire safety performance.

Semi-Quantitative Embodied Carbon Impact

The impact will vary depending on the structure type, where some structures may be able to eliminate fireproofing and others may only allow a modest or minimal reduction. While fireproofing material is embodied carbon intensive, it typically represents a relatively small amount of material overall in the building. Therefore, the potential embodied carbon reduction in archetype studies ranges from 0% to 3% for the overall structure.

Next Steps and Potential Roadblocks

Potential Next Steps

1. Identify representative structural archetypes for analysis.
2. Perform computational modeling and experimental validation of structural fire performance.
3. Compare predicted performance with current prescriptive requirements.
4. Identify potential opportunities for modest revisions to prescriptive fireproofing provisions.
5. Develop guidance documents supporting the use of performance-based structural fire design.

Potential Roadblocks

Several barriers may slow adoption of improved fireproofing provisions:

- The fire safety community tends to be appropriately conservative due to the life-safety implications of fire events.
- Fire testing programs are expensive and require significant industry funding.
- Performance-based methods require specialized expertise that is not uniformly available across the profession.
- Effective implementation requires coordination among structural engineers, fire protection engineers, and code officials.

Collaboration between research institutions, professional organizations, and code development bodies will be essential. Potential partners include the Charles Pankow Foundation, AISC, ICC, and fire protection engineering organizations.

Summary

Continued research and education could enable gradual improvements to fireproofing provisions that maintain life safety while improving efficiency and reducing environmental impacts.

PCC8: REVAMP CONCRETE MIX PROPORTIONING

Introduction

The main driver for carbon emissions related to concrete mixtures is the proportion of cement used to achieve strength and other defined performance parameters. ACI 318 defines requirements for previous strength test performance to establish a statistical basis for acceptance of concrete mixtures. This priority code change includes the exploration of amending the current mix proportioning requirements and acceptance criteria to reduce the amount of strength overdesign, and thus cement, needed for concrete mixtures.

Current Code Provisions

ACI 318-25 26.4.3.1(b) requires that concrete mixture proportions be established in accordance with Article 4.2.3 of ACI SPEC-301-20.

ACI 301 sets f'_{cr} requirements, which are the required average compressive strength of test data to demonstrate that concrete mix designs are capable of delivering the defined f'_c. The f'_{cr} requirements are set by either by equations based on historic test data, or they are set values above f'_c if historic data is not available. As the commentary of ACI 318 notes, the probability of not meeting the acceptance criteria is not more than 1 in 100 by following the proportioning requirements of ACI 301.

The quality control requirements in ACI 301 include conformance to ASTM C94. Certification must also be confirmed by one of the following: certified by the NRMCA Program for Certifications of Ready Mixed Concrete Facilities, approval by a state highway agency, or certification by an equivalent program that audits for conformance with ASTM C94.

Potential Change

In order to meet the proportioning requirements and to reduce risk of not meeting the specified strength, concrete mixes often include larger proportions of cementitious materials than needed. Possible code changes to reduce the code required overdesign include Options A-C, described below.

Option A: Re-evaluate the current ACI 301 requirements and determine if the current requirements could be altered and continue to provide a probability of exceedance of 1%.

- Where f'_{cr} requirements in ACI 301 are determined using historic sample standard deviation: f'_{cr} requirements for this condition are determined by the standard deviation of groups of consecutive strength test results and a k-factor determined by the number of tests.

- Where f'_{cr} requirements in ACI 301 are determined without historic strength test data: f'_{cr} requirements for this condition are determined by a tabulated required overstrength based on the concrete strength classification. For example, 1200 psi overstrength is required for concrete strengths between 3000 psi and 5000 psi. A study could be made of concrete mixture test data to determine if these set overstrength values produce the expected probability of exceedance.

- It is also necessary to evaluate if overstrength is driven by suppliers trying to avoid a singular data point not meeting strength. This evaluation may indicate whether alternative acceptance criteria based on average strength or another adjustment could be made.

Option B: Implement expanded QA/QC procedures, beyond the those currently used, that may reduce the scatter of test results and allow lower overdesign values to be used.

- This option would require determining how the current quality control procedures and testing requirements could be expanded and the impact of these changes.

- Research about the statistical distribution of strength test results for different levels of quality control or testing requirements would be needed. This could also include studying the impact of increased frequency of testing on overdesign values.

- This option may require additional testing or research to compare the results from different QA/QC programs and testing frequency.

Option C: Adjust the probability of exceeding the acceptance criteria to a value slightly higher than 1%. Values explored could include 1.1%, 1.25%, or 1.5%.

- This would require researching the history of how the current 1% requirement was set and how the current statistical evaluation of mixes could be adjusted to yield a different %.

- Required average strength (f'_{cr}) tables in ACI 301 would be adjusted to lower values.

None of the options listed above address the overdesign driven by risk avoidance, where concrete strengths provided can be significantly higher than the f'_{cr} required. While this can be a major driver of overdesign, reducing overdesign driven by risk avoidance would be difficult to codify.

Technical Background and Literature

ACI Committee 301 – Specifications for Concrete Construction most recent publication was ACI SPEC-301-20 with the next planned publication as ACI SPEC-301-26. The ACI 301 committee has several subcommittees, with the ACI 301-0D Concrete Mixtures subcommittee in charge of the pertinent section on the overstrength required for concrete mixtures.

Key References

- American Concrete Institute. (2020). Specifications for concrete construction (ACI SPEC-301-20).

- American Concrete Institute. (2025). *Building code for structural concrete – Code requirements and commentary* (ACI CODE-318-25).
- ASTM International. (2023). *Standard specification for ready-mixed concrete* (ASTM C94).
- National Ready Mixed Concrete Association. (2010). *Concrete in practice 9: Low concrete cylinder strength*.
- National Ready Mixed Concrete Association. (2014). *Technology in practice 2: Establishing the required average strength of concrete mixtures*.

Potential Impact of Change

A re-evaluation of the testing and acceptance criteria for concrete mixtures has the potential to impact several aspects of the efficiency, cost, and consistency of concrete construction.

Material Efficiency and Embodied Carbon

Provisions related to acceptance criteria could reduce the amount of cementitious material required in concrete mixtures to provide an equivalent specified design strength. Since cementitious materials are the largest contributor to embodied carbon in a concrete mixture any reduction in cementitious material, and specifically cement, is likely to achieve reductions in embodied carbon.

While many factors influence the overall concrete mix design, any reduction to the required statistical overstrength, or improvements in quality control, will help to ensure that when compressive strength is a specified for a mixture, cementitious content is used efficiently.

Cost Reduction

Cementitious materials are the most expensive component of a concrete mixture. Any reduction in the required cementitious material content, to meet a given specified compressive strength, would provide cost benefits.

Improved Consistency in Material Testing

Option B listed in this section would study approaches to improving the consistency of testing outcomes for concrete mixtures. A higher level of consistency in test results would provide a higher level of confidence in concrete mixture performance.

Semi-Quantitative Embodied Carbon Impact

The impact varies depending on the structure type and class of concrete utilized for the majority of design, although the potential code change could be significantly impactful because it could affect all concrete mixes in a given building. For example, if the structure utilizes primarily 3,000 psi concrete, there is limited ability or benefit to reducing required strength and therefore embodied carbon further. However, structures design with concrete in more typical classes like 5,000 psi could be significantly impacted by this potential code change. Therefore, the potential embodied carbon reduction in archetype studies ranges from 5% to 12%.

Next Steps and Potential Roadblocks

The level of research required and potential roadblocks vary for the options listed.

Option A Potential Next Steps and Roadblocks

- Study the historic performance of sample concrete mixture strength test data against the expected probability of exceedance.
- Identify alternative approaches to the determination overstrength for trial mixtures and acceptance criteria to be studied. A potential challenge here may be that existing requirements are well established, so alternatives may be difficult to implement.
- Perform a statistical analysis to show the probability of exceedance from alternative approaches and estimate the value of carbon avoided.

Option B Potential Next Steps and Roadblocks

- Research alternate approaches to quality control procedures that impact the consistency of strength test outcomes.
- Determine possible changes to existing typical quality control requirements and their impact to reliability outcomes. Implement testing to determine statistical distribution of performance outcomes based on these changes. Estimate the value of carbon avoided due to the lower f'_{cr} values. This would likely require extensive research on quality control measures beyond the current standards and the amount of testing to provide sufficient data would be large. The study would also need to account for differences in materials and procedures across regions where the code is implemented.

Option C Potential Next Steps and Roadblocks

- Gather history of how current 1% probability of exceedance requirements were set.
- Identify alternate probability percentages to be studied and perform statistical analysis to determine f'_{cr} values associated with alternate probabilities and estimate the value of carbon avoided.
- Although this research is the most limited of the options considered, it would require that code committees and AHJs adopting the code accept a higher probability of concrete mixtures not meeting their strength performance requirements. Accepting higher risks of not meeting the anticipated strength could be a substantial barrier to code change.

Quantification of Embodied Carbon Reduction

In all options, the proportion of reduction in cement resulting from the lower f'_{cr} in the options listed would need to be quantified. This reduction could then be applied across the volume of concrete for a building or region to determine overall impact. It is also recommended that the impact of the user imposed overstrength beyond the code-based requirements be quantified and included in any study of this topic.

Need for Coordination

It should be noted that the current ACI 318 code committee is considering changes to the section of the code that references the ACI 301 concrete mixture proportioning requirements. Any adjustments made by the current ACI code committee will need to be considered.

Summary

Cement content in concrete mix is often driven by minimum strength requirements for cylinder tests as part of the quality control processes. Reconsideration of the basis for these minimum strength requirements could produce significant reductions in embodied carbon.

PCC9: REVAMP PRESCRIPTIVE CONCRETE DURABILITY REQUIREMENTS

Introduction

To ensure the durability of concrete, the ACI 318 applies prescriptive requirements that impact the amount of cement required for concrete mixtures. There are alternative approaches to providing durability that may allow for lower amounts of cement to be used that could replace the existing prescriptive requirements. This topic examines whether changing or providing alternative approaches to concrete durability could reduce the carbon impacts of concrete mixtures while maintaining the required level of durability.

Current Code Provisions

ACI 318-25 Chapter 19 defines the durability requirements for concrete mixtures, which are controlled by the exposure class assigned to the member. These exposure classes are assigned by the licensed design professional based on the anticipated severity of exposure for the structural member. Exposure categories include freeze/thaw, sulfate exposure level, water contact conditions, and corrosion protection of reinforcing from chlorides.

ACI 318 Table 19.3.2.1 lists requirements for concrete by exposure class. These requirements include maximum water-cement ratio, minimum f'_c, air content, acceptable cement types, limits on admixtures, and maximum water-soluble chloride ion content. Of these requirements, the prescriptive minimum strength in particular has the potential to directly impact cement content of concrete mixtures by potentially requiring strengths greater than needed for design.

It should be noted that there have already been recent changes in ACI 318 regarding durability requirements. Prior to the publication of ACI 318-25, Table 19.3.2.1 limited the amount of supplementary cementitious material (SCM) that could be used for Exposure Class F3 mixes. ACI 318-25 removed Exposure Class F3 and the SCM limits related to that exposure class. Thus, one of the previous durability requirements that had an impact on the amount of Portland cement in concrete mixes has already been removed in the current ACI 318 building code.

Potential Change

Alternative approaches to providing durability could include:

- The use of supplementary cementitious materials to reduce permeability of concrete within defined conditions.
- Requirement for testing to prove desired durability characteristics rather than prescriptive requirements on water-cement ratio and compressive strength. There are several existing

ASTM test methods that can be used to determine characteristics such as resistance to chloride ion penetration, concrete electrical resistivity, sulfate resistance of mortars, and the susceptibility of concrete to damage by freezing and thawing.

- Allowance for less stringent water-cement ratios or compressive strengths if either sealers or additives that provide durability are included in the concrete mixtures.
- Establish alternative pathways for addressing freeze/thaw durability issues beyond defining air content. Concrete mixtures with required high air contents often require proportionally more cementitious materials than comparable non-air mixes.

Technical Background and Literature

ACI has both an informational committee (ACI 201 - Durability of Concrete) and a code committee (ACI 321 - Concrete Durability Code) focused on durability. ACI 201's published ACI PRC-201.2-23 Durable Concrete - Guide, includes information on the exposure types and factors impacting concrete durability. The ACI 321 code committee is currently developing a code including both prescriptive and performance-based approaches to designing concrete mixtures for durability characteristics.

There are a number of ASTM tests that have the possibility to replace or be used as an alternative pathway to demonstrating durability for the different exposure types, including exposure to freeze/thaw, sulfates, water, and chlorides.

Key References

- American Concrete Institute. (2023). Durable concrete – Guide (ACI PRC-201.2-23).
- American Concrete Institute. (2025). Building code for structural concrete – Code requirements and commentary (ACI CODE-318-25).
- ASTM International. (2024) Standard test method for microscopical determination of parameters of the air-void system in hardened concrete (ASTM C457).
- ASTM International. (n.d.). Standard test method for resistance of concrete to rapid freezing and thawing (ASTM C666).
- ASTM International. (2024). *Standard test method for length change of hydraulic-cement mortars exposed to a sulfate solution* (ASTM C1012).
- ASTM International. (2025). *Standard test method for electrical indication of concrete's ability to resist chloride ion penetration* (ASTM C1202).
- ASTM International. (2025). *Standard test method for bulk electrical resistivity or bulk conductivity of concrete* (ASTM C1876).

Potential Impact of Change

The durability provisions of the code are applied to any members exposed to outside elements that have the potential to degrade the structure over time. Adjusting the existing code provisions has the potential to impact the following:

Alignment With Performance-Based Design

Providing performance-based alternatives to the prescriptive requirements for durability in ACI 318 allows concrete mixture designs to directly provide the required durability characteristics. By approaching durability design in way that enables specific measurements of the desired characteristic, outcomes can be more reliable and consistent.

Material Efficiency and Embodied Carbon

For exposed concrete elements where durability provisions, including water/cement ratio, strength, and/or air content, drive cementitious contents higher than otherwise required, alternative durability pathways may result in a reduction in cementitious materials. The embodied carbon of concrete mixtures is largely driven by the overall amount of cementitious materials and using performance-based durability approaches rather than prescriptive has the potential to reduce these high impact materials. For concrete mixes that may already be utilizing a proportion of supplementary cementitious materials that may provide reduced permeability, the availability of an alternative permeability compliance pathways may allow less Portland cement to be used.

Cost Savings

Lowering the overall required amount of cement has the potential to reduce costs.

Semi-Quantitative Embodied Carbon Impact

The impact varies depending on the structure type, class of concrete utilized, and exposure and durability requirements for each concrete mix design. Therefore, the potential embodied carbon reduction in archetype studies ranges from 1% to 5%.

Next Steps and Potential Roadblocks

The approach to durability design has evolved over time and there are several potential opportunities for additional changes.

Potential Next Steps

Further research is required to determine if there are already alternative approaches developed and what alternatives have sufficient research to support change proposals for building code requirements.

Quantification of the embodied carbon impact of the proposed changes is needed. The structural member types most likely to be impacted to are exterior elements with higher exposure categories, including but not limited to:

- Foundation or basement walls extending above grade with snow or ice
- Exterior structural slabs
- Elements exposed to deicing salts or seawater

Of these structural members, elements that may have minimum size and reinforcing requirements that would not require the f'_c values required by Table 19.3.2.1 may be driven to include higher amounts of cementitious materials than would otherwise be needed. To quantify the impact of this change, an estimate of how often concrete mixtures are specified with higher f'_c than required would be needed.

Potential Roadblocks

A potential roadblock to implementation may be if alternative permeability or performance testing is too costly or time consuming, it may not be widely implemented enough to impact carbon.

Summary

Prescriptive durability requirements could be improved or replaced with performance-based approaches, that could reduce the necessary cement content to meet these requirements, reducing embodied carbon.

PCC10: REFINE EFFECT OF CONCRETE SLAB STRENGTH ON COLUMN AXIAL FORCE TRANSMISSION

Introduction

In cast-in-place concrete construction, it is common practice for columns to be designed with higher specified compressive strength concrete than the surrounding floor slabs. Columns carry significantly higher axial stresses and therefore often benefit from higher concrete strengths to control column dimensions and improve structural efficiency.

At slab-column interfaces, the supporting slab must resist localized bearing stresses produced by the column load. When the slab concrete strength is substantially lower than the column concrete strength, the allowable bearing stress in the slab may control design.

In practice, this condition may lead to column puddling, where higher-strength slab concrete is placed locally around the column. Alternatively, the specified compressive strength of the slab concrete may be increased across the entire floor plate to avoid localized detailing or specification.

While these approaches satisfy the current design provisions, they introduce construction complexity and may increase material usage. This topic examines whether current design and acceptance practices, particularly the reliance on 28-day compressive strength as the controlling metric, may lead to conservative outcomes that increase material quantities without materially improving structural safety.

Current Code Provisions

Concrete design provisions limit the allowable bearing stress where columns bear on slabs or foundations. These provisions are contained in ACI 318 Section 15.8.1.

In practical design situations, the relationship between column concrete strength and slab concrete strength often becomes a controlling factor. When the slab concrete strength is significantly lower than the column concrete strength, the allowable bearing stress in the slab may be insufficient without additional measures.

Industry practice has therefore developed around an approximate rule that when the slab concrete strength is less than about 70% of the column concrete strength, additional detailing measures are commonly introduced.

Typical responses include:

- Column puddling, where higher-strength concrete is placed locally around the column.

- Increasing slab concrete strength across the entire floor plate.

Because slab concrete strength is typically defined and accepted based on 28-day compressive strength, this ratio often governs detailing decisions even though concrete strength continues to increase beyond 28 days.

Potential Change

One potential change would be to provide clearer guidance within ACI provisions or commentary that allows designers to consider later-age concrete strength when evaluating localized bearing conditions at slab-column interfaces.

Possible approaches could include:

- Allowing later-age compressive strength values (e.g., 56-day strength) to be used when evaluating slab-column bearing stresses.
- Adjusting the axial force transmission through slab provision to directly account for anticipated slab strength at maturity.
- Providing further guidance for multi-age concrete strength specifications, where both 28-day and later-age strengths are defined.
- Allowing project teams to demonstrate adequate bearing capacity through documented mix performance and testing data.

These approaches would not modify the existing bearing equations but would clarify how concrete strength may be evaluated in situations where structural loads are applied gradually and concrete strength continues to increase over time.

Technical Background and Literature

Concrete strength development is a time-dependent process. While compressive strength is commonly specified at 28 days, cement hydration continues beyond this age, resulting in measurable increases in strength over time. For many structural concrete mixtures, compressive strength at 56 days or later may exceed the 28-day strength by a meaningful margin.

In building construction, column loads are typically applied gradually as floors are cast sequentially and as construction progresses upward. Full design loads at slab-column interfaces often occur well after the slab concrete has reached and exceeded its 28-day compressive strength.

The current practice of evaluating slab-column bearing based strictly on the 28-day strength may therefore not fully reflect the strength of the concrete at the time when peak loads are applied.

Similar considerations regarding later-age concrete strength have been explored in other areas of structural concrete practice, including:

- Performance-based concrete specification
- Later-age strength acceptance for mass concrete and durability considerations
- Structural design provisions that recognize strength development over time

These observations suggest that additional guidance regarding the consideration of later-age concrete strength at localized bearing regions may be warranted.

Key References

- American Concrete Institute. (2025). *Building code for structural concrete – Code requirements and commentary* (ACI CODE-318-25).
- Bianchini, A., Woods, R, and Kesler, C. (1960). Effect of Floor Concrete Strength on Column Strength. *ACI Structural Journal*, 56(11).

Potential Impact of Change

Providing clearer guidance on the use of later-age concrete strength could reduce the need for column puddling or global increases in slab concrete strength. Potential impacts include:

Reduced material use

Avoiding increases in slab concrete strength across entire floor plates when localized bearing checks govern.

Lower embodied carbon

Reducing required strength and, therefore, cement content in slab mixes may lower the embodied carbon of concrete, particularly for large floor areas.

Improved constructability

Avoiding localized puddling regions simplifies concrete placement and coordination during construction.

Better alignment with structural behavior

Recognizing later-age concrete strength may better reflect the actual conditions under which loads are applied in cast-in-place construction.

Semi-quantitative embodied carbon impact

The impact will be more significant on high-rise archetypes where high strength concrete column mixes are typically used for efficiency, although this may allow wider use of high strength concrete columns in structures where not typically used which could offer additional embodied carbon reductions. Given that this potential code changes affects the slab concrete mix embodied carbon, and the slab concrete quantity is typically more than 50% of the concrete in the structure, the potential embodied carbon reduction in archetype studies ranges from 3 to 10%.

Next Steps and Potential Roadblocks

Potential Next Steps

- Reviewing available research on concrete strength development and bearing behavior.
- Evaluating the frequency with which column puddling or slab strength increases occur in practice.
- Assessing the embodied carbon implications of slab strength increases driven by localized bearing requirements.
- Developing example design cases comparing traditional approaches with later-age strength verification.

Potential Roadblocks

- The industry's strong reliance on 28-day compressive strength as the standard acceptance metric.
- Industry and practitioner concerns regarding testing, documentation, and verification of later-age strength.
- A preference for simple and predictable code enforcement practices.
- Limited familiarity among project teams with multi-age concrete specifications.

Addressing these challenges would likely require clear guidance, documentation, and industry education to ensure that any changes maintain the reliability and safety of current design practice.

Summary

Providing clearer guidance on the use of later-age concrete strength to satisfy bearing stress requirements could reduce the need for column puddling or global increases in slab concrete strength, reducing embodied carbon in floor systems.

PCC11: REDUCE EARLY STRENGTH REQUIREMENTS FOR POST-TENSIONED SLABS

Introduction

In cast-in-place concrete construction, post-tensioned slabs are frequently used as floor systems for their efficiency as the post-tensioning allows for thinner slabs and less reinforcement compared to a traditional mild-reinforced slab. Because of the thinner slabs and therefore less concrete quantity required, these slabs inherently can reduce embodied carbon significantly compared to mild-reinforced slabs. However, due to the construction sequencing requirements for stressing of these tendons and the corresponding minimum concrete compressive strength needed before stressing, many of the potential embodied carbon benefits are mitigated.

Post-tensioned slabs are designed via ACI 318 for strength and serviceability criteria. The serviceability criteria address stress limits at service level stresses, as well as establishing allowable stress limits at transfer (post-tensioned tendon stressing). Allowable stress at transfer is evaluated based on initial concrete strength, which is typically assumed to be 3,000 psi. Additionally, local and general anchorage zones for the post-tensioning anchors are evaluated based on initial concrete strength.

Due to construction sequencing requirements and construction schedule requirements, it is often desired to stress post-tensioned tendons as soon as 48 hours after the concrete has been placed. This puts pressure on the concrete mix design to reach 3,000 psi at 48 hours, which often requires a much higher cementitious quantity compared to a more traditional slab mix. Additional conservatism (higher cement content) in the mix design has been observed due to construction schedule pressure, given the consequence of schedule delay if initial strength is not met.

While the current design approach works, it may unnecessarily be driving cement content and embodied carbon in post-tensioned slab mix designs. Therefore, this potential code change examines whether there is ability to change initial strength requirements for post-tensioned slabs.

Current Code Provisions

Allowable stress limits for post-tensioned slabs are established in ACI 318 24.5.3, which include compressive and tensile stress limits based on initial concrete strength (f'_{ci}). The tensile limits can be exceeded where additional bonded reinforcement is applied.

Anchorage zones for post-tensioned tendons are designed in accordance with ACI 318 25.9. This addresses the local forces that occur directly behind the anchorage device as well as the general zone where the concentrated post-tensioning force is transferred to the concrete and distributed more uniformly through the cross section.

Monostrand unbonded tendons, which are used commonly in slab construction, are more directly addressed in ACI SPEC-423.7. This document establishes strength requirements for anchorages at transfer load based on f'_{ci} and the area of the anchorage. While this document does not establish a minimum strength, industry practice has established the use of f'_{ci} of 3,000 psi for anchorage design.

ACI 318 25.9.4.5.4 also establishes a minimum strength of 2500 psi for stressing of single-strand tendons.

Potential Change

One potential change is to establish an alternative initial concrete strength requirement for anchorage zone design to permit lower initial strength. For example, changing from industry standard practice f'_{ci} of 3,000 psi to 2,500 psi would result in lower cementitious content for post-tensioned slab mixes. This may require redesign and testing of anchorage zones to establish adequate designs.

Other potential changes could include establishing an appropriate tolerance on initial strength to reduce concrete mix overdesign. Additionally, test methods to establish initial strength could be reconsidered to reduce uncertainty and improve reliability such to allow more efficient mix designs.

Technical Background and Literature

Concrete strength development is time dependent, and early strength (1-3 days after placement) often carries much uncertainty. While ACI 318 currently allows anchorage design for 2,500 psi, industry standard practice has adopted 3,000 psi for initial compressive strength requirements. This is documented in reports such as ACI 423.3R-17 which acknowledges that post-tensioning anchor assembly suppliers often specify a minimum compressive strength of 3,000 psi.

Key References

- American Concrete Institute. (2017). Recommendations for Concrete Members Prestressed with Single-Strand Unbonded Tendons (ACI 423.3R-17).
- American Concrete Institute. (2014). Specification for Unbonded Single-Strand Tendon Materials (ACI SPEC-423.7).

Potential Impact of Change

Lower cement content and embodied carbon

Reducing initial strength requirements can have a direct reduction on cementitious content in post-tensioned slab mixes and therefore directly reduce embodied carbon. Given that slab concrete often makes up more than 50% of the concrete in a cast-in-place concrete building, the potential impact is significant.

Semi-Quantitative Embodied Carbon Impact

The impact will be limited to post-tensioned concrete structures, although could have a significant reduction on those structures given the slab concrete represents the majority of the concrete in most buildings. Based on archetype studies, the potential embodied carbon reduction ranges from 3% to 10%.

Next Steps and Potential Roadblocks

Potential Next Steps

- Evaluate existing post-tensioning anchor designs for impacts associated with reduced initial compressive strength.
- Evaluate post-tensioned slab mixes for impacts associated with reduced initial compressive strength.
- Test post-tensioned slab anchors with lower initial strength.

Potential Roadblocks

- Industry reluctance to reduce initial strength based on concerns of premature anchorage failure.
- Reluctance to change existing anchorage designs.
- Concern regarding other associated risks of lower initial concrete strength such as long-term performance related to deflection, serviceability, and impacts of early construction loading.

Addressing these challenges will require research and testing to establish safe working limits for initial concrete strength for post-tensioned slab construction, as well as industry guidance and education to establish new safe and reliable design practices.

Summary

Design provisions that enabled post-tensioning anchor designs for reduced initial compressive strength could reduce embodied carbon by reducing required strength, and therefore cement content.

OTHER POSSIBLE CHANGES

Reevaluate Seismic Design Factors

Introduction

Seismic Force Resisting Systems (SFRS) are designed for safety based on the following criteria: For structures posing an ordinary risk, the conditional probability of collapse of all or a significant portion of the structure will not exceed 10 percent given the occurrence of a "Risk (reliability) - targeted Maximum Considered Earthquake" ground motion (MCE_R). The limit on the conditional probability is intended to apply to a population of similar structures within the same class of SFRS. Under current standards, the reliability target is a 1 percent chance of failure in 50 years, except in the highest seismic hazard areas. Given this, the mean recurrence interval for the MCE_R ground motions in much of the US is in the range of 2000 years, or more. A proposal to change the target to 1.5 percent and eliminate the semi-deterministic cap that governs the highest hazard areas is moving through the standardization process and will most likely be the basis of the ground motions in the 2028 edition of ASCE 7. The performance at the MCE_R accepts significant damage to the structure, generally limited to ductile deformations. Structures for which the consequence of failure is more serious are designed to be somewhat stronger to reduce the amount of damage and the likelihood of collapse.

Much of the design of buildings for seismic loads is done based upon a linear response analysis in which the seismic demand is estimated based upon adjustment factors that account for acceptable inelastic performance in very rare events. Some significant structures in high seismic hazard regions are designed based upon nonlinear response history analysis.

Current Code Provisions

Linear design procedures allow reduction of the internal force effects (strength demands) computed via linear elastic procedures with a Response Modification Factor, ***R***, intended to account for a system's ductility and expected overstrength. It is used as a divisor, so SFRS with larger ductility and larger overstrength have larger values for ***R***. The values of ***R*** factors for established SFRS were originally based on empirical performance, elementary analysis, and professional judgement. A more rigorous procedure has been developed and published as FEMA P-695, and the ***R*** factor and other design parameters for many of the more highly ductile systems have been validated by this procedure.

There are also stiffness requirements that limit the lateral drift, and these criteria can control structural proportions for some types of flexible structures, especially where the ground motions are large. An additional seismic performance factor, $\mathbf{C_d}$, is used in the drift calculation.

Provisions are under development for a second basic criterion: functional recovery. The basis for these provisions is that structures should be restored to a basic level of functionality following a significant ground motion in a target period of time (hours, days, weeks, or months) that depends on the importance of that structure's function to the community. These provisions will appear in the 2026 NEHRP Provisions, and it is likely that these provisions will appear as a nonmandatory appendix in the 2028 edition of ASCE/SEI 7 *Minimum Requirements for Design Loads and Associated Criteria for Buildings and Other Structures*. The ground motion for this criterion is less than the MCE_R motions. There is a strength criterion, based upon a smaller value of reduction from linear elastic response, i.e. $\mathbf{R_{fr}} < \mathbf{R}$, but the limits on lateral drift are much more strict than those considered in strength based design, and those limits will control the design of many SFRS where they apply.

Rationale for Not Recommending Further Consideration

This issue cannot be placed in the high priority set of issues for reduction of embodied carbon in building construction for two primary reasons:

- Seismic design criteria have evolved significantly over the past few decades. Among these changes, the pending move towards functional recovery criteria may increase the design requirements. As a result, it may not be likely that reevaluation of seismic performance factors would reduce design requirements or embodied carbon.
- Validating the seismic design parameters is costly and time consuming, as described below.

Rather, it is possible that increased use of performance-based design (the nonlinear response history analysis) and increased use of supplemental damping would be a more effective way to reduce the effect of seismic design on embodied carbon.

Technical Background and Literature

The current basis for validating seismic design paraments for a new SFRS are laid out in *Quantification of Building Seismic Performance Factors* (FEMA Publication P695, June 2009, available at https://nehrpsearch.nist.gov/static/files/FEMA/fema_p695.pdf). The methodology requires analytically testing a suite of archetypes using incremental dynamic analysis for their collapse performance when exposed to a suite of ground motions. The analytical response models must be based upon extensive physical testing, and the archetypes are designed to follow the requirements of ASCE/SEI 7 as well as the design and detailing rules specifically created for the new SFRS. The process for a new SFRS is expensive and time-consuming, particularly because of the necessary physical testing to develop the basis for the response models.

First generation functional recovery design provisions will be available in the 2026 version of the NEHRP Recommended Seismic Provisions.

Opportunities for Other Standards and Guideline Documents

This effort focused primarily on ASCE 7, ACI 318, and AISC 360. However, the committee also identified a number of other opportunities for code changes that could reduce embodied carbon.

Two significant areas where further consideration of embodied carbon would be helpful are:

- **Existing buildings, and the International Existing Building Code**. Renovation and reuse of existing buildings represents a major opportunity for reduction of embodied carbon, as it reduces the embodied carbon impact of constructing new buildings. However, some of the provisions related to existing buildings may be excessively conservative. Additionally, the threshold for structural retrofits is not consistent across many jurisdictions and can present a roadblock for projects that might otherwise consider re-use of an existing building. This represents a major opportunity for further consideration. We recommend that a similar examination to that conducted here related to the IEBC be considered for embodied carbon reduction strategy.
- **Changes needed to typical practice.** In many cases, embodied carbon is driven by conservative building practices that lead to overdesign well in excess of code requirements. This can be as simple as practices of 'rounding up' to the next typical size, or excessively smoothly or simplifying designs. We recommend an effort to provide evidence-based guidance documents that address some of these practices to support designers and others in reducing that conservatism.

Some of the specific ideas discussed, but not pursued further, are listed in the Appendix.

CONCLUSIONS: ROADMAP FOR RESEARCH AND CODE CHANGE

This report reexamines the primary U.S. structural building codes and standards through the lens of embodied carbon, systematically rethinking some of the fundamental engineering design requirements to identify the potential for embodied carbon reduction.

The resulting top 11 potential code changes **represent a roadmap for future research and action that can lead to reduction in embodied carbon in the built environment** without compromising safety, durability, or serviceability of structures. We believe these topics are **achievable in the near-term through targeted, funded research to develop data and recommendations for code change proposals.**

This roadmap provides action items for researchers, funders, and members of building codes and standards committees to produce code change proposals for review by codes and standards committees.

- **Researchers and engineering practitioners** are encouraged to use these ideas as a starting point to develop a deeper technical basis and evidence for the proposed changes. We anticipate that careful study of some of these topics may offer further opportunities for carbon reduction. This evidence should include a more detailed life cycle assessment of the potential embodied carbon impacts. The end result of each research effort should be an actionable, carefully-supported code change proposal.
- **Funding** is needed to support these research efforts to enable timely development of the necessary technical basis to support code change proposals.
- **Collaboration with members of codes and standards committees** is essential to ensure that research efforts anticipate and investigate possible concerns.

Structural engineers should take the lead on innovations in embodied carbon reduction in building structures given their outsized impact on specifying the materials and design requirements. Changes to the building codes to inherently reduce embodied carbon is one of the most basic tools to produce broad, near-term impact.

APPENDIX. LIST OF ALL PROVISIONS CONSIDERED

This Appendix provides a summary of the committee's brainstorming exercise of provisions that may provide an opportunity to reduce embodied carbon. As such, they reflect the views of the committee. They are provided here for consideration by others, recognizing that, for those not described in detail in the report, more careful examination is warranted.

ACI 318 Provisions Considered

Item	Title	Section(s)	Description	Potential Impact
1	Revamp Durability Requirements	Chapter 19, 19.3.2.1	The current durability requirements are prescriptive, particularly where there is a prescriptive minimum strength. A more direct measurement / assessment of durability to achieve the same performance could be provided in lieu of or as an alternative to prescriptive methods.	Reduction in prescriptive w/c ratios and f'_c requirements would reduce usage of cementitious material
2	Revamp Concrete Mix Proportioning	26.4.3.1, ACI 301 4.2.3	Current provisions in ACI 301 require 'overdesign' of concrete compressive strength, particularly when limited historical test data is available. Some of this overdesign is based on historical distribution of material strength that may not reflect current practice. Additionally, concrete suppliers that take additional QA/QC steps may be able to further reduce the overdesign.	Reduction in mix 'overdesign' would lead to direct reduction of cementitious material
3	PT High Early Strength	ACI 423.7	Typical practice requires 3,000 psi strength before stressing PT tendons. Often this is targeted at 1 or 2 days after pouring the slab, which drives the cementitious content in PT slab mixes. Stressing at a lower strength may be technically possible subject to proving safety and serviceability are met, although may require changes to PT anchor sizing.	Reduction in high early strength requirement would lead to direct reduction of cementitious material
4	Maximum f'c limits	22.5.3.1, 18.10.4.1	Reconsider maximum limits on f'_c related to shear strength of members, especially shear walls. This criterion drives proportioning of many elements, and higher strength material is more common that can exceed these limits, although may require further research and testing.	Increasing f'_c limits would permit reduction in proportioning of some members, resulting in less concrete material usage
5	Special Shear Wall Optimization	18.1	Reconsider many aspects of special shear walls that may be conservative. For example, PMM design based on conservative load combinations (not expected) drive rebar proportioning. As an example, coupling beam designs are often too strong to permit energy dissipation.	Reduction in reinforcement and concrete proportions of typical special shear walls.
6	Test Cylinders	26.12	Many 4 x 8 cylinders are taken for a project which leads to a not insignificant amount of material, in addition to mix 'overdesign' to compensate for poor handling of	Reduction in concrete quantity required for cylinders, potential for reduced use of

			cylinders. Consider allowing alternative strength measurement metrics such as maturity meters.	cementitious material for mix 'overdesign'
7	Two-way Slab Thickness Criteria	8.3.1.1	Minimum slab thickness may be conservative for some areas, especially for lightly loaded structures.	Reduced quantity of concrete and reinforcement
8	Slab Detailing Requirements	8.7	Some prescriptive slab detailing requirements may not be necessary based on results of finite element analysis. Reduction of elimination of these requirements should be considered.	Reduce quantity of reinforcement used
9	Column Puddling Requirements	15.8, 18.10.6.4(h)	Current code requires puddling when floor system is less than 0.7 f'_c of column to allow bearing stress transfer. We know this is conservative based on late maturity strength gain. In many cases, these forces result in slabs poured with 'inflated' f'_c to eliminate puddling requirements. Similar requirements exist for shear wall boundary elements which may be similarly conservative.	Reduce quantity of cementitious material required
10	Minimum Integrity Requirements	8.7.5.6	Slab integrity bar is based on joint dimensions. Allowing other methods to determine this reinforcement would save rebar.	Reduce quantity of reinforcement used
11	Minimum Hook Development Requirements	25.4.3.1	Minimum hook development of 6 inches controls some designs, for example, curb or wall dowels into slabs on metal decks. A reduced hook embedment, even if at a capacity reduction, would not force unnecessary thickening or additional material usage.	Reduce cases where slab or member is artificially thickened, resulting in reduction of concrete quantity
12	Increase Use of High Strength Reinforcement	20.2.2.4	While the code has greatly increased ability to use high strength reinforcement, lack of research has limited its use in specific cases. This includes shear, non-special structural walls, axial, etc.	Reduce reinforcement quantity, and potentially member proportioning resulting in concrete quantity reduction
13	ϕ factors	21.2.1	Reconsider all phi factors, including revisiting design target reliability.	Reduce quantities of concrete and rebar, including member proportioning reductions
14	Air content requirements	NA	Air content requirements in Lightweight Concrete - minimum air content in lightweight concrete is governed primarily by the prescriptive UL Listing for fire ratings. Air content in lightweight concrete is at odds with requirements for finish durability.	Reduction in concrete quantities
15	Concrete requirements with respect to frost depths	19.3.1	Consider F0 and F1 as defined in ACI 318 19.3.1. Requirement for air content to handle frost requirements can drive cement content. These requirements may affect an entire foundation system, but only a portion of it is actually above the frost depth.	Reduce prescriptive concrete mix requirements (strength and air entrainment) to reduce cementitious material quantities

16	Grade 80 reinforcement requirements	25.4.2.2	With bar cutoffs in those systems, you have a bar on the top (or bottom) developing for about 80% of the beam length. This means that where you previously might have had stirrups at say 12" you now need them to be at 8" solely for this provision. This not only increases the bar tonnage but more importantly the pieces and labor. Without this provision gr 80 would be the clear choice	Reduce reinforcement requirements when using high grade reinforcement
17	Concrete breakout strength for design of anchorage	Section 17.6.2	·Section 17.6.2 lays out the requirements for the concrete breakout strength of anchors in tension. These requirements are excessively conservative. A BSSC PUC Issue Team a couple of cycles ago identified the problem to be the breakout strength was determined using a very conservative cutoff of the data resulting in extremely low breakout strengths compared to actual experience. ACI chose not to act on the conclusions of the Issue Team.	Increase in concrete breakout strength will reduce or eliminate additional anchor reinforcement.
18	Minimum reinforcement; one way shear in non-prestressed members	Table 22.5.5.1	It comes down to Av, min for slabs, walls, and wide beams where previous code versions allowed exceptions for these elements because they redistribute shear differently. Leads to increase in basic basement walls from 8 inch to 10 in, but more significant increases elsewhere	Recent change that increases wall and slab sizes

ASCE 7 and IBC Provisions Considered

Item	Category	Title	Section(s)	Description	Potential Impact
1	General	Target reliabilities	Chp.1	Investigate target reliabilities in Chapter 1, including load factors; compare against empirical data and other risks. Risk targets may be unnecessarily conservative and methods for determining whether risk targets are met may also be unnecessarily conservative	These target reliabilities affect all of the loads, so adjustments to lower reliability or conservatism could be very impactful.
2	Live loads	Re-evaluate live load values in table	Chp. 4; Section 4.3.1; table 4.3-1	Live loads have not been widely validated using statistical approaches and accounting for modern use/applicability	There is potentially significant conservatism in the live loads that are very commonly used. If addressed for live loads that affect the design of lots of buildings and carbon intensive material systems, it would have significant effects.
3	Live loads	Office	Chp. 4; Section 4.3.1; table 4.3-1	Review traditional widely used live loads for modern applicability	
4	Live loads	Residential	Chp. 4; Section 4.3.1; table 4.3-1	Review traditional widely used live loads for modern applicability	
5	Live loads	Partition	Chp. 4; Section 4.3.1; table 4.3-1	Consider more flexible assumptions for interior partition weights.	
6	Live loads	Roof	Chp. 4; Section 4.3.1; table 4.3-1	Review traditional widely used live loads for modern applicability	
7	Live loads	Storage	Chp. 4; Section 4.3.1; table 4.3-1	Review traditional widely used live loads for modern applicability	
8	Live loads	Light and heavy manufacturing	Chp. 4; Section 4.3.1; table 4.3-2	Review traditional widely used live loads for modern applicability	
9	Live loads	Assembly spaces	Chp. 4; Section 4.3.1; table 4.3-3	Re-evaluate the 100 psf live load applied in many places, including public spaces in residential buildings	
10	Live loads	Live loads on non-building structures	Chp. 4	Revisit these loads as they are used for industrial structures. In many cases maintenance loads, rather than values for light/heavy manufacturing would be appropriate	
11	Live loads	Construction Loads		Assess if current assumptions reflect modern construction practices or are overly conservative.	

12	Live loads	Live load reductions	Section 4.7 and Table 4.7-1	Different live load reduction factors for different occupancies; current approach has been developed mostly based on office occupancy. The current reduction rules have remained mostly unchanged for decades. They may not reflect actual usage or risk and do not consider a holistic approach to system design.	When not fully utilized, live load reductions result in overdesigned members and frames, especially in large-floorplate office, lab, or parking structures. This directly leads to increased concrete and steel usage. Impacts most mid- to high-rise buildings and structures with large tributary areas. Updates could apply to slabs, beams, girders, and columns across many occupancies.
13	Live loads	Modify live load factor	Chp. 2; Chapter 4	1.6 factor is not uniformly conservative; develop risk-based loads considering appropriate level of risk for gravity loads. Factors may differ for different occupancies to provide consistent reliability, as well as difference in consequences of failure. Consider also the 0.5 factor on roof live loads when combined with floor live loads.	There is potentially significant conservatism in the live loads that are very commonly used. If addressed for live loads that affect the design of lots of buildings and carbon intensive material systems, it would have significant effects.
14	Live loads	Performance Based Live Load Determination	Chp 2 & 4	Reconsider fundamentally how we determine live loads by using a performance-based approach. Perform stochastic analyses of all major live loads to determine the full distribution of maximum loads as a function of return period. Use these to determine values for maximum considered live load and risk-targeted maximum considered live load and design live load, and then review the load factors and load combination factors on a comprehensive, consistent evaluation predicated on target reliability levels and other criteria, one of which is embodied carbon. Factors may differ for different occupancies to provide consistent reliability, as well as difference in consequences of failure. Live load reduction could be included if incorporated into the probability analysis for different influence areas	Potential to remove significant conservatism from all live loads. Broad potential impact but is a more ambitious change to pursue.

15	Dead loads	Dead load factor	Chp. 2; Chp. 3	Review reduction of dead load factors in load combinations for uplift loads. Consider if uplift/ rocking is an ok behavior under design level EQ.	Overturning can govern the design of foundations.
16	Dead loads	Self-weight and superimposed dead loads	Chp. 2; Chp. 3	Study separating self-weight and superimposed as independent loads with potentially different load factors, not lumped together. Weight of concrete/steel may not require a 1.2-1.4 factor	Potential reduction in load factor on some aspects of dead loads could have big impacts on a large number of buildings
17	Soil Loads	Load factor on soil hydrostatic loads	Chp. 2	Study load factors and combinations with hydrostatic loads	Could reduce demands in some cases.
18	Soil Loads	Buoyancy	Section 3.2	Combination of buoyancy uplift with seismic is overly conservative.	Could reduce demands in some cases.
19	Load combinations	Load combinations for special structures such as stadiums	Chp. 2	Consider load combinations and spatial distribution of these loads for very large structures (i.e., the likelihood that an entire structure is fully loaded)	Could have substantial influence on material requirements for these large special structures
20	Risk categories	Transition from Risk Category II/III	Table 1.5-1 and IBC	Clarify how occupancy is to be determined in determining whether the 5000-occupant limit is reached	Clarify cases where loads need to be increased for Risk Category III, potentially reducing loads.
21	Seismic	Minimum base shear	12.8	Re-evaluate the 1% minimum value	Consider whether this minimum could be relaxed in some cases, reducing seismic forces
22	Seismic	Vertical seismic loading	12.4.2.2	Expand the Exception 2 of 12.4.2.2 allowing vertical seismic load effect Ev to be taken as 0 to more elements. In particular, yielding elements like vertical reinforcement in shear walls are often governed by low gravity [(0.9-0.2Sds) *D+E] load combinations even if they aren't particularly sensitive to vertical ground motions. Foundations would be another element where removing the vertical seismic effects may help reduce unnecessary conservativism.	Reduction in member proportioning and reinforcement requirements in many cases where vertical seismic load effect unnecessarily controls the design.
23	Seismic	Height limits	Table 12.2-1 & Commentary	From the Commentary -- >Height limits for seismic force-resisting systems have existed in the code for over 50 years and have evolved incrementally. However, the specific numeric values - especially the sudden drop	These limits can prevent use of lighter structural systems (e.g., light-frame wood, braced steel) in taller buildings in SDC D and higher, even where detailing or nonlinear

				from “unlimited” to “not permitted” between SDCs - are still based on subjective engineering judgment rather than fully calibrated performance data.	analysis might allow safe performance. This leads to more material-intensive systems being used instead.
24	Seismic	Seismic design factors (R, Ω_0, Cd)	Table 12.2-1 & Commentary	From the Commentary -->The R, Ω_0, and Cd factors assigned to structural systems are foundational to seismic design, but many were originally based on engineering judgment and limited testing in the 1970s. Although some updates have occurred, many values still lack rigorous calibration or consistent physical interpretation. Affects all seismic designs nationwide; especially impactful in SDC C, D, and higher. Influences both new construction and retrofit strategies.	These factors affect seismic base shear and displacement calculations, influencing lateral system sizing, diaphragm demands, and foundation loads. Conservative or inconsistent values can result in heavier-than-necessary systems or discourage selection of more efficient ones. If study found these values are overly stringent, could reduce demands in seismic areas.
25	Seismic	Seismic reserve capacity of gravity systems	Table 12.2-1 and general	Gravity framing system beams to column connections can provide measurable lateral resistance. Can we leverage this capacity especially in lower SDCs? Extensive research shows some of these connections offer meaningful stiffness and strength. Widely applicable to low- and moderate-seismic regions across the U.S. including most of the central and eastern U.S. Potential for broad adoption in retrofit and new design.	Adding lateral systems increases the use of concrete, steel braced frames, shear walls, and footings that may not be necessary. Capturing existing capacity could reduce new structural material needs.
26	Seismic	Seismic irregularities	12.3	Evaluate material cost implications of irregularity triggers and redundancy factors.	Are these triggering extra design requirements where they are not needed?
27	Seismic	Seismic redundancy factor	12.3	Seismic Design Categories D, E, and F require a minimum of two lines of lateral resistance in each direction and impose a 30% load penalty (ρ = 1.3) for non-redundant systems. Are the requirements overly prescriptive? Evaluate material cost implications of irregularity triggers and redundancy factors.	Very high, especially for schools, offices, housing, and small commercial buildings in SDC B/C zones across the U.S. These provisions can drive unnecessary braced bays, shear walls, collectors, or foundations - even in systems that meet drift and strength limits. This results in

					material-heavy layouts and higher embodied carbon.
28	Seismic	Seismic accidental eccentricity	12.3	Requirements for consideration of accidental eccentricity increase design effort and material. Has been reconsidered recently	If study showed accidental eccentricity is triggered when not needed, could reduce seismic design impacts.
29	Seismic	Seismic natural period and Cu factor	Section 12.8.2.1 and Table 12.8-1; Commentary C12.8.2.1	From the Commentary -->The empirical period equations are based on regression fits from historical building data. Cu factor caps (from Table 12.8-1) are not derived from reliability theory or modern performance data, but from expert judgment to prevent underestimation of seismic base shear. Impacts nearly all lateral system designs across low- and moderate-seismic regions. Applies across building typologies and materials. Offers potential for major reductions in conservative overdesign.	Overly conservative base shear (due to short period estimates or capped periods) can lead to increased member sizes and foundation demands. This is particularly impactful in short and mid-rise buildings, which make up a large share of the built environment.
30	Seismic	Low/mid seismic	Chp. 12	Ensure high seismic design is not unfairly placed on low/ mid seismic regions, including ground motion assumptions, fragility curves, etc. Focus on ductility and remove other unnecessary requirements.	Seismic design can have major impacts in material/member design and selection, even in areas that are not high seismic.
31	Seismic	Seismic detailing; R = 3 systems	Table 12.2-1 & general seismic detailing	Only structural steel has a clearly defined R=3 system with no special seismic detailing required. Other materials (e.g., concrete, masonry, wood) must use systems with required seismic detailing even in low/moderate zones (SDC B/C), potentially adding unnecessary material, cost, and complexity.	Seismic detailing often increases member sizes, reinforcement, proprietary systems, and labor-intensive construction. In many regions, this detailing may provide limited additional safety. Could have very high impacts, especially for schools, offices, housing, and small commercial buildings in SDC B/C zones across the U.S.
32	Flood	Flood load factors	Chp. 5	Provide guidance on capping hydrostatic factors at grade.	Reduce overdesign of below grade walls.
33	Serviceability	Gravity deflection limits	Appendix C	Deflection criteria (e.g., L/360, L/240) are not mandatory but widely adopted. Values are based on historical precedent and visual perception, not rigorous empirical data. For	Commonly results in oversizing beams, joists, and slabs - particularly long-span or shallow members. This increases material

				many structural types, especially slabs, these limits drive up stiffness requirements unnecessarily. Appendix CC explicitly states that these are not based on occupant comfort studies or damage thresholds.	volume and embodied carbon even when strength is adequate.
34	Serviceability	Load combinations for deflection	Appendix C	Current code applies full design loads (e.g., 1.0L) to deflection checks, but commentary references potential use of reliability-based approaches (e.g., load with 0.05 annual probability of exceedance). This is not formalized in the code.	Using full design loads for serviceability often controls member stiffness, especially for slabs. Realistic load combinations could justify smaller members. Affects most systems
35	Serviceability	Vibration Limits	Appendix CC	The code does not specify performance-based vibration criteria for floors. Appendix CC notes that vibration limits are typically handled outside of code by specialty consultants. Without calibrated or standardized targets, engineers may overdesign for perceived safety.	Overdesigning vibration - especially in long-span composite steel and concrete slabs - can significantly increase material volume. Applies to education, healthcare, lab, and office buildings with long-span or vibration-sensitive spaces. Could reduce design in less vibration sensitive places.
36	Serviceability	Long term Deflection and Creep	Appendix C / Commentary (CC)	ASCE 7's commentary allows creep multipliers (e.g., 2x or 3x) based on material type and moisture conditions, but these are often applied conservatively without clarity on actual performance or measured deflections. Long-term limits (e.g., L/240 for D+L) can control concrete and wood systems. Commentary notes that creep multipliers are highly dependent on exposure and construction sequencing. Default factors may be overly conservative. Greater guidance on actual long-term deflection performance - or use of monitoring data - could help reduce unnecessary material use while preserving serviceability.	Leads to larger member depths and heavier reinforcement or section sizes. In concrete slabs or timber floors, deflection often governs before strength - especially for longer spans and thinner profiles. Affects concrete and timber systems across multifamily, office, institutional, and renovation projects.
37	Serviceability	Deflections under snow loads	Appendix CC	Appendix CC suggests using 20-year MRI (5% exceedance) snow load for serviceability checks, but in practice, most	Applying higher MRI loads (50-year) to serviceability limits can cause oversizing

				engineers use the same snow load as for strength (50-year MRI). This could lead to inconsistency and potentially excessive member sizes. (20-year MRI is about 80% of 50-year MRI). Impacts of roof framing in all snow regions; particularly high impact in schools, healthcare, and large-span roof projects.	for snow-governed elements like roof beams, decks, or trusses. A clear, enforceable 20-year MRI guideline could reduce embodied carbon while meeting performance goals.
38	Other	Functional recovery (low damage design)	General	Encourage design that balances performance goals and material efficiency through prescriptive and performance-based approaches.	Reduction in damage greatly reduces carbon costs of rebuilding after disaster events.
39	Other	Slab restraint and IBC/fire engineering	IBC	Prescriptive requirements that can govern design in many cases. Thinking on slab restraint in IBC could be more line with ACI-216 (i.e. typical PT slabs can be considered restrained).	Prescriptive requirements with large impact on design and embodied carbon.

AISC 360 Provisions Considered

Item	Category	Title	Section(s)	Description	Potential Impact
1	General	ϕ factors and target reliabilities	AISC 360, B3.1	Recalibrate where justified by performance data.	We may be able to reduce unnecessary conservatism.
2	Serviceability	Vibration	AISC Design Guide 11	Examine conservatism in serviceability criteria, especially for low-occupancy zones. Consider revised limits in non-sensitive applications to reduce material use	Reduced material usage with smaller sections or less impactful assemblies
3	Detailing	Connection detailing	AISC 360 J10 and G2	Study stiffener and reinforcement default requirements for simplification.	Reduced material usage and reduced welding
4	General	Steel overstrength	AISC 360, C1.4	The difference between Fy and Fye for steel materials leads to substantial overstrength in steel lateral systems that forces larger columns and beam sizes. For example, steel tube braces are designed to assume Fy, but then the rest of the system is designed for Fye. Commentary notes that specified minimum Fy is used even though most mill products exceed this by 5 - 10%. No provisions allow for using actual tested values unless in material test-specific situations.	Better coordination between steel supplier and designer could drastically reduce the difference between these two values. Prevents taking advantage of excess strength in mill steel, which could reduce member sizes. High for most steel framed structures.
5	General	Width to thickness ratio	AISC 360-22. B4.1, Table B4.1b	Revisit compactness and slenderness criteria that govern member sizing, specifically for low-load applications. Many of the width-to-thickness limits are based on historical judgment, semi-empirical calibration, or conservatism rather than robust modern statistical calibration or recent test data. Modern materials, fabrication tolerances, and improved QA/QC may justify updates.	These are currently under revision (balloting for AISC 341-27). May lead to heavier sections when conservative limits push members into lower classifications (e.g., compact to noncompact). High for most steel framed structures.
6	General	Slenderness ratio	AISC 360-22. E7, E3	The compression limits of 200 and tension limit of 300 are not clearly derived from reliability analyses; commentary suggests they reflect long-standing practice and minimum stiffness expectations. May be overly conservative for certain cases.	May cause rejection of members that would otherwise perform adequately, increasing member size and material usage. High for most steel-framed structures.
7	Seismic	Axial load in braced frame columns	AISC 341 F1	Axial load in braced frame columns is not a well understood phenomenon and is drastically overestimated in simple designs for buildings	Reduced section sizes and reduced material usage

				more than a couple stories in height.	
8	Seismic and Wind	Performance-based design	AISC 341 Appendix 1	Develop "procedural PBD" approach for specific systems to replace capacity design, with goal of allowing PBD without peer review via a prescriptive approach	Reduced section sizes and reduced material usage
9	General	Higher modulus steel	AISC 360 A3	Applications for higher modulus steel in stiffness-driven applications	Potential to reduce member sizes broadly and use less material
10	General	Higher strength steel	AISC 360 A3	Opportunities for up to 100 ksi steel in columns and other axial members	Ongoing interest at AISC in this topic.
11	General	Damping	N/A	Prescriptive methods to facilitate damper use	Potential to reduce member sizes of lateral force resisting system, or exceed typical heights for current systems
12	General	Fireproofing	Appendix 4	Appendix 4 provides a pathway for performance-based fire design (PBFD) of structural steel, allowing evaluation of real fire scenarios, compartmentation, and inherent structural capacity under fire. However, it is not mandated by AISC 360, not referenced in IBC or ASCE 7, and not accepted as a substitute for prescriptive fireproofing in most jurisdictions.	Prescriptive fireproofing methods (e.g., cementitious spray, enclosures) add significant embodied carbon. When PBFD is not allowed or considered, steel designs often require unnecessary material or protective coatings even where structural performance under fire could be demonstrated without them. High impact on most mid- and high-rise steel buildings subject to ASTM E119 fire rating requirements. Affects schools, offices, healthcare, residential, and lab typologies.
13	General	Metal deck slab shear strengths	AISC 360 Section I	Underestimating the strength of slabs increases reinforcing, f'c and/or thicknesses.	Reduce gauge of metal deck, reduce thickness of concrete topping slabs, and reduce reinforcement requirements

Current Practice Recommendations

Title	Description
Stiffeners	Provide more guidance on implementation of stiffeners to reduce conservative practice.
Building reuse	Building reuse is a huge opportunity to reduce carbon emissions. Commentary on the carbon impacts of new vs reuse could be included to illustrate the opportunities for reduced emissions. This also brings up the question of the loads in the IEBC and ASCE41: are the performance expectations the same as new and can the design provisions in the codes for existing buildings be adjusted similar to what is being suggested for new building codes?
Concrete mix development	Promote faster acceptance of optimized low-carbon mixes.
Geotechnical conservativeness	Bearing stresses allowance directly affects foundation sizes, but what really happens when bearing is exceeded? Do they fail or is the reality of slightly larger settlements. Clarity, maybe further understanding, and varying of this could offer savings. Similarly, lateral seismic components on basement walls are huge - yet there is little history of problems that i know of.
Lateral analysis and modeling	Further guidance/acceptance for including soil and foundation flexibility to avoid overly conservative fixed based assumptions.
General concrete design strategy	Design optimization strategies to minimize concrete quantity while increasing reinforcement generally led to more carbon-efficient designs. Examples include spread footings with or without shear reinforcement, etc. If the design practitioners had better guidance on these strategies, there could be a significant impact without changing code.

REFERENCE LIST

American Concrete Institute. (1992). Prediction of creep, shrinkage, and temperature effects in concrete structures (ACI PRC-209-92(08)).

American Concrete Institute. (2017). Recommendations for concrete members prestressed with single-strand unbonded tendons (ACI 423.3R-17).

American Concrete Institute. (2020). Report on deflection of nonprestressed concrete structures (ACI 435R-20).

American Concrete Institute. (2020). Specifications for concrete construction (ACI SPEC-301-20).

American Concrete Institute. (2023). Durable concrete – Guide (ACI PRC-201.2-23).

American Concrete Institute. (2025). Building code for structural concrete – Code requirements and commentary (ACI CODE-318-25).

American Concrete Institute. (2014). Specification for unbonded single-strand tendon materials (ACI SPEC-423.7).

American Concrete Institute Committee 435. (1966). Deflections of reinforced concrete flexural members (ACI 435.2R-66). *ACI Journal Proceedings*, 63(6).

American Institute of Steel Construction. (2022). Specification for structural steel buildings (ANSI/AISC 360-22).

American Society of Civil Engineers. (2022). Minimum design loads and associated criteria for buildings and other structures (ASCE 7-22).

American Society of Civil Engineers. (2020) Performance-based structural fire design: Exemplar designs of four regionally diverse buildings using ASCE 7-16 (Appendix E).

ASTM International. (2022). Standard specification for cold-formed welded carbon steel hollow structural sections (HSS) (ASTM A1085/A1085M-22).

ASTM International. (2023). Standard specification for ready-mixed concrete (ASTM C94).

ASTM International. (2024). Standard test method for microscopical determination of parameters of the air-void system in hardened concrete (ASTM C457).

ASTM International. (2026). Standard test method for resistance of concrete to rapid freezing and thawing (ASTM C666).

ASTM International. (2024). Standard test method for length change of hydraulic-cement mortars exposed to a sulfate solution (ASTM C1012).

ASTM International. (2025). Standard test method for electrical indication of concrete's ability to resist chloride ion penetration (ASTM C1202).

ASTM International. (2025). Standard test method for bulk electrical resistivity or bulk conductivity of concrete (ASTM C1876).

Bailey, C. G. (1999). The structural behavior of steel columns during a compartment fire in a multi-story braced steel-frame. *Journal of Constructional Steel Research,* 52(2).

Bergendahl, M. (2021). Experimental investigation to compare the cyclic response of A500 and A1085 HSS braces (MS thesis, University of Washington).

Bianchini, A., Woods, R, and Kesler, C. (1960). Effect of Floor Concrete Strength on Column Strength. *ACI Structural Journal*, 56(11).

Branson, D. E. (1965). Instantaneous and time-dependent deflections on simple and continuous reinforced concrete beams (HPR Report No. 7, Part 1). Alabama Highway Department, Bureau of Public Roads.

Branson, D. E. (1970). Discussion of "Proposed revision of ACI 318-63: Building code requirements for reinforced concrete." *ACI Journal Proceedings*, 67(9).

Branson, D. E. (1971). Compression steel effect on long-time deflections. *ACI Journal Proceedings*, 68(8).

Branson, D. E. (1977). Deformation of concrete structures. McGraw-Hill.

Branson, D. E., Meyers, B. L., & Kripanarayanan, K. M. (1970). Time-dependent deformation of noncomposite and composite prestressed concrete structures. In *Symposium on Concrete Deformation, Highway Research Record* 324 (pp. 15–43). Highway Research Board.

Brockenbrough, R. L. (2003). MTR survey of plate material used in structural fabrication, *AISC Engineering Journal,* 1st Quarter.

Corotis, R., Hooper, J., & Klemencic, R. (2023). Design live loads for office gathering spaces. *Journal of Structural Engineering,* 149(12).

Corotis, R., Arwade, S., & Ha, K. D. (2025). Safe and sustainable parking garage live loads in the age of the electric vehicle (RGA #05-24, Final Report). Charles Pankow Foundation.

Corotis, R. (1985). Probability-based design codes. *Concrete International*, 7(4).

Culver, C. G. (1976). Survey results for fire loads and live loads in office buildings (National Bureau of Standards Special Publication 85). National Bureau of Standards.

Ellingwood, B., Galambos, T., MacGregor, J., & Cornell, C. A. (1980). Development of a probability based load criterion for American National Standard A58 (National Bureau of Standards Special Publication 577). National Bureau of Standards.

Ellingwood, B. (1989). Serviceability guidelines for steel structures. *AISC Engineering Journal*, 26(1).

Galambos, T. V., & Ellingwood, B. (1986). Serviceability limit states: Deflection. *Journal of Structural Engineering*, 112(1).

Galambos, T. V. (1973). Structural deflections: A literature and state-of-the-art survey (Building Science Series Vol. 47). US National Bureau of Standards.

Galambos, T. V. (1978). Load and resistance factor design, *AISC Engineering Journal*, 3rd Quarter.

Gernay, T. (2024). Performance-based design for structures in fire: Advances, challenges, and perspectives. *Fire Safety Journal*, 142.

Ghali, A., & Favre, R. (1986). Concrete structures: Stresses and deformations. Chapman and Hall.

Hasofer, A. M. (1968). Statistical model for live floor load. *Journal of the Structural Division*, 94(ST10).

ISO (2015). General principles on reliability of structures. 4th Edition.

Kaldestad, J. (2022). Experimental investigation into the cyclic response of A1085 HSS braces (MS thesis, University of Washington).

Liel, A. B., DeBock, D. J., Harris, J. R., Ellingwood, B. R., & Torrents, J. M. (2017). Reliability-based design snow loads. II: Reliability assessment and mapping procedures. *Journal of Structural Engineering*, 143(7).

Liu, J. (2016). Updates to expected yield stress and tensile strength ratios for determination of expected member capacity in the 2016 AISC seismic provisions, *AISC Engineering Journal*, 53(4).

Liu, J., Sabelli, R., Brockenbrough, R. L., & Fraser, T. P. (2007). Expected yield stress and tensile strength ratios for determination of expected member capacity in the 2005 AISC seismic provisions, *AISC Engineering Journal*, 44(1).

Liu, J. (2021). AISC steel structures research update: Structural fire engineering. *AISC Engineering Journal*, 58(4).

Luco, N., Ellingwood, B. R., Hamburger, R. O., et al. (2007). Risk-targeted versus current seismic design maps for the conterminous United States. In *SEAOC 2007 Convention Proceedings*. Structural Engineers Association of California.

McAllister, T. P., Wang, N., & Ellingwood, B. R. (2018). Risk-informed mean recurrence intervals for updated wind maps in ASCE 7-16. *Journal of Structural Engineering*, 144(5).

Mitchell, G. R., & Woodgate, R. W. (1970). Floor loadings in office buildings – The results of a survey (CP3/71). Building Research Station.

National Institute of Standards and Technology. (2010). Best practice guidelines for structural fire resistance design of concrete and steel buildings (NIST Technical Note 1681).

National Ready Mixed Concrete Association. (2014). Concrete in practice 9: Low concrete cylinder strength.

National Ready Mixed Concrete Association. (2014). Technology in practice 2: Establishing the required average strength of concrete mixtures.

Post-Tensioning Institute. (2025). Post-tensioned structural concrete (ACI/PTI 320-25).

Swensson, K. D. (2012). Guide to the design of building systems for serviceability: In accordance with the 2012 IBC and ASCE/SEI 7-10. International Code Council; National Council of Structural Engineers Associations.

Tsay, W.-Y., & Corotis, R. (1983). Probabilistic load duration model for live loads. Journal of Structural Engineering, 109(4).

West, M., Fisher, J., and Griffis, L. (2004). Serviceability design considerations for steel buildings. American Institute of Steel Construction (Steel Design Guide No. 3, 2nd ed.).