A comparative study of plain and reinforced concrete isolated footings: structural behavior, constructability, and embodied carbon

by

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Abstract

A major challenge confronting architecture, engineering, and construction (AEC) industry is the threat of climate change – an issue to which the industry itself is a significant contributor. As a result, AEC professionals carry a responsibility to improve material efficiency and reduce embodied carbon across common structural systems. One opportunity lies in re-evaluating isolated spread footings, a ubiquitous foundation element in low-rise buildings throughout the United States of America. These footings, typically composed of concrete and reinforcing steel, transfer gravity loads through concrete compression and soil bearing while also resisting incidental shear and moment demands. Although concrete alone can resist axial, shear, and flexural forces to a limited degree, standard practice relies on reinforcing steel to provide adequate shear and flexural capacity.

This report examines the *ACI 318-19(22)* code provisions on plain concrete, reviews design requirements for reinforced foundations, and compares the capacities of plain versus reinforced spread footings subjected to concentrated axial loads of 50, 100, 200, and 300 kip concentric axial loads. Key site and material parameters such as allowable soil bearing capacity (2,000 psf, 3,000 psf, and 4,000 psf) and concrete compressive strength (3,000 psi and 4,000 psi) are varied to enable direct comparisons. To evaluate environmental implications, the increased concrete volume required for plain concrete design is assessed relative to the reduced use of reinforcing steel. It was found that when the ultimate axial load is 50 kips or smaller, plain concrete footings have a reduced environmental impact conversely when the ultimate axial load is 100 kips and greater, reinforced concrete footings have a greater material efficiency that yield a lower embodied carbon. Finally, the report discusses broader constructability and

sustainability considerations, including procurement, transportation, and installation impacts associated with each footing type.

Table of Contents

List of Figures viii
List of Tables ix
Acknowledgements x
Chapter 1: Introduction 1
Chapter 2: Historical Development of Concrete Design Standards and Construction Practices 4
2.1 Formation of Early Bodies and the Origins of Concrete Standardization 4
2.1.1 Development of U.S. Concrete Design Standards 4
2.1.2 Evolution of ACI 318: From Working Stress to Ultimate Strength 6
2.2 Formwork Design and Construction 10
2.2.1 Overview and Function 11
2.2.2 Design Responsibility and Documentation 11
2.2.3 Site Preparation and Layout 12
2.2.4 Formwork Materials and Configuration 13
2.2.5 Assembly, Alignment, and Bracing 13
2.2.6 Application of Release Agents 14
2.2.7 Concrete Placement and Curing 14
2.2.8 Form Removal and Backfilling 15
2.3 Regional Design Considerations and Impacts on Material Acquisition 15
2.3.1 Historical Context of Aggregate Sourcing in the Midwest 16
2.3.2 Evolution of Extraction and Processing Technologies 16
2.3.3 Environmental and Regulatory Considerations 17
2.3.4 Sustainable Practices and Local Sourcing 17
2.3.5 Future Directions 18
2.4 Construction Equipment Impact 19
2.4.1 Emission Sources and Equipment Characteristics 19
2.4.2 Sustainable Equipment Practices and Technological Advancements 20
2.4.3 Economic and Operational Constraints 21
2.4.4 Integrated Planning and Emission Reduction Strategies 22
2.5 Factors of Safety in Structural Design 22

2.5.1 Material Variability and Construction Tolerances 23
2.5.2 Geotechnical Uncertainty and Comparative Safety 24
2.5.3 External Loads and Extreme Events 24
2.5.4 Reliability-Based Design and Statistical Interpretation 25
2.5.5 Discussion 25
Chapter 3: Parametric Study on the Size and Composition of Spread Footings 27
3.1 Overview 27
3.1.1 Soil Conditions 28
3.1.2 Building Framing and Column Loading 29
3.1.3 Concrete Strength and Type 30
3.1.4 Footing Dimensions and Formwork Considerations 32
3.1.5 Reinforcing Steel 36
3.2 Plain Concrete Design of Isolated Shallow Footing Example 37
3.2.1 Design Overview and Input Parameters 38
3.2.2 Determination of Footing Plan Area 39
3.2.3 Factored Soil Pressure and Ultimate Moment 39
3.2.4 Nominal Moment Capacity of Plain Concrete 40
3.2.5 Shear and Bearing Checks 41
3.2.5.1 One-Way and Two-Way Shear 41
3.2.5.2 Bearing Strength 42
3.2.6 Discussion 42
3.3 Reinforced Concrete Design of Isolated Shallow Footing 43
3.3.1 Design Parameters and Dimensional Adjustments 43
3.3.2 Reinforcing Steel Design and Layout 44
3.3.3 Flexural Strength and Utilization 45
3.3.4 One-Way Shear Strength 45
3.3.5 Two-Way (Punching) Shear Strength 46
3.3.6 Bearing Strength 46
3.3.7 Summary and Comparison Notes 47
3.4 Code Analysis of the Plain Concrete Design Example 47
3.4.1 Scope and Governing Documents 47

3.4.2 Critical Sections and Where They Come From 47
3.4.3 Plain Concrete Design Path and Adjustment from Reinforced Section Design 48
3.4.4 Reinforced Concrete Path (Chapters 13, 7, 8, and 22) 50
3.4.5 Practical Implications for the Parametric Study 51
Chapter 4: Results 53
4.1 Study Overview 53
4.2 Concrete Volume and Depth Relationships 54
4.2.1 Plain Concrete Footings 54
4.2.2 Reinforced Concrete Footings 56
4.3 Reinforcing Steel Quantities and Efficiency 58
4.4 Shear Performance Analysis 59
4.4.1 One-Way Shear 59
4.4.2 Two-Way (Punching) Shear 59
4.5 Bearing Strength Evaluation 60
4.6 Plain Concrete Performance 60
4.7 Comparative Summary and Discussion 61
Chapter 5: Recommendations and Conclusions 64
5.1 Calculation Methodology 64
5.2 Influence of Concrete Volume and Strength 65
5.3 Role of Reinforcing Steel 66
5.4 Construction Methods and Formwork Considerations 67
5.5 Regional Material Sourcing and Transportation 69
5.6 Comparative Summary of Embodied Carbon Results 70
5.7 Discussion: Structural Efficiency as a Sustainability Tool 72
5.8 Future Implications 72
5.9 Summary 73
References 75
Appendix A - Plain Concrete Design Example 82
Appendix B - Reinforced Concrete Design Example 87
Appendix C - Calculation Result Tables 94

List of Figures

Figure 2-1 - Illustration of Typical Foundation Formwork Configuration for an Isolated Spread Footing 15
Figure 4-2 - Plain Concrete Footing Volumes vs. Applied Ultimate Load 56
Figure 4-3 - Reinforced Concrete Footing Volumes vs. Applied Ultimate Load 57

Appendix Figure A.1. Page 1: Base Criteria 82
Appendix Figure A.2. Page 2: Bending Design Check 83
Appendix Figure A.3. Page 3: One-way Shear Design Check 84
Appendix Figure A.4. Page 4: Two-way Shear Design Check 85
Appendix Figure A.5. Page 5: Bearing Design Check 86
Appendix Figure B.1. Page 1: Base Criteria 87
Appendix Figure B.2. Page 2: Bending Design Check 88
Appendix Figure B.3. Page 3: Bending Design Check cont. 89
Appendix Figure B.4. Page 4: Development Length Design and Pad Size Check 90
Appendix Figure B.5. Page 5: One-way Shear Design Check 91
Appendix Figure B.6. Page 6: Two-way Shear Design Check 92
Appendix Figure B.7. Page 7: Bearing Design Check 93
Appendix Figure C.1. Plain Concrete – Material Results 94
Appendix Figure C.2. Plain Concrete – Moment Capacity Results and Utilization 95
Appendix Figure C.3. Plain Concrete – One-way Shear Capacity Results and Utilization 96
Appendix Figure C.4. Plain Concrete – Two-way Shear Capacity Results and Utilization 97
Appendix Figure C.5. Reinforced Concrete – Material Results 98
Appendix Figure C.6. Reinforced Concrete – Moment Capacity Results and Utilization 99
Appendix Figure C.7. Reinforced Concrete – One-way Shear Capacity Results and Utilization 100
Appendix Figure C.8. Reinforced Concrete – Two-way Shear Capacity Results and Utilization 101
Appendix Figure C.9. Parametric Study Results - 100 kip Ultimate Loading 102
Appendix Figure C.10. Plain vs. Reinforced Concrete Section – 25 kip Ultimate Loading 103

List of Tables

Table 3-1 Parametric Study Examples – Site and Material Definitions 38
Table 4-2 Full Results Highlights and Overview 61
Table 5-3 Embodied Carbon Values of Materials and Transport 65
Table 5-4 Average Embodied Carbon Values – All Cases Calculated 70
Table 5-5 Embodied Carbon Totals of Plain and Reinforced Sections 71

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Chapter 1: Introduction

Modern construction faces an unprecedented challenge—understanding and mitigating the environmental consequences of human activity on a rapidly changing climate. Mounting scientific evidence has compelled industries worldwide to reassess their practices to reduce their environmental impact. Within the Architecture, Engineering, and Construction (AEC) sector, a variety of analytical tools and strategies and tools have emerged to evaluate the ecological footprint of buildings throughout their life cycles (Intergovernmental Panel on Climate Change [IPCC], 2021); (World Green Building Council [WGBC], 2023).

A central focus of sustainability evaluation in construction lies in quantifying the cumulative impact of materials used across a project's lifespan. Specialized calculators, such as the Embodied Carbon in Construction Calculator (EC3) and life-cycle assessment (LCA) tools, enable practitioners to quantify the quantity, source, and environmental burden of building materials (Building Transparency, 2025); (International Organization for Standardization, 2006). A crucial metric in determining the ecological footprint of an action is the assessment of carbon emissions during the construction phase. Specifically, in the realm of structures, this impact is termed "embodied carbon," encapsulating the greenhouse gas emissions generated during the extraction, production, transportation, and installation of construction materials (American Institute of Architects (AIA), 2019); (American Society of Civil Engineers (ASCE), 2023).

Structural systems, constituting a fundamental aspect of buildings, typically exhibit relatively limited variability across projects. The primary structural materials include reinforced concrete, structural steel, timber, brick, and concrete block masonry with ancillary materials such as grout, anchors, bolts, reinforcing steels, aggregates, and cementitious compounds also

contributing to a building's embodied carbon footprint (Cabeza, Boquera, Chafer, & Verez, 2021).

Despite efforts to revolutionize material optimization and production processes, the most effective strategy to reduce the embodied carbon of a structure remains trimming excess material use (Pomponi, 2018). Excessive design practices are commonly found in construction, notably in the use of generic details, particularly when applied across diverse projects with various loading parameters that may be lower than the initially designed capacity. Moreover, overly conservative interpretations of code standards contribute to inflated designs (Kibert, 2022). These instances often stem from attempts to streamline the design process for cost-saving purposes or to enhance on-site constructability.

While such intentions may seem positive initially, they can have unforeseen consequences. Neglected additional costs or environmental impacts not accounted for in the initial assessment could escalate when multiplied across numerous instances. Repetition of these practices on multiple projects can exacerbate the issue, potentially compromising the overall sustainability of the project and its environmental impact (Silvius & Schipper, 2014).

Therefore, this report concentrates on the impact of repetitive design practices on a structure's embodied carbon. Specifically, it delves into the correlation between spread footings and their role in standard low-rise structures. The analysis revolves around the context of a one to three-story building, focusing on interior columns bearing floor and roof loads over a conceptualized tributary area. For the purpose of this study, no reductions in design will be considered, and the relevant loading parameters will be precisely applied within this defined scope.

For this study, embodied carbon is assessed from the point of material procurement, transportation to the manufacturing plant, and the manufacturing process. Some groups provide information about transport and installation at a project site, however those vary in consistency of data input, and should be relied on sparingly. This boundary definition, commonly referred to as a cradle-to-gate assessment, provides a comprehensive understanding of the carbon emissions expected for the structural system as it is in the design process (European Committee for Standardization, 2011); (International Organization for Standardization, 2006).

Chapter 2: Historical Development of Concrete Design Standards and Construction Practices

This chapter examines the evolution of standards governing concrete design in the United States, from the early 20th century to the present. It contextualizes how advances in materials science, structural mechanics, and construction practice have shaped successive code revisions. It also discusses key practical aspects, such as formwork, aggregate sourcing and handling, heavy-equipment use, and design safety factors that interact with design standards and influence concrete's role in structural systems.

2.1 Formation of Early Bodies and the Origins of Concrete Standardization

The evolution of concrete design standards in the United States reflects more than a century of progress in engineering practice, material science, and professional collaboration. Before formal codification, early 20th-century construction practices often relied on empirical knowledge and regional expertise, resulting in inconsistent quality, safety, and performance. As industrialization accelerated and infrastructure demands grew, the need for standardized guidance became increasingly urgent.

2.1.1 Development of U.S. Concrete Design Standards

The formal standardization of concrete design in the U.S. traces back to the founding of the National Association of Cement Users (NACU) in 1904, whose mission was to disseminate knowledge and coordinate among engineers, architects, and educators in the cement/concrete industry (American Concrete Institute, n.d.). Establishing NACU marked a crucial milestone in developing concrete technology in the United States. From its inception, NACU prioritized disseminating information and fostering collaboration among engineering, architecture, construction, and education professionals involved in concrete-related endeavors. Through an

early emphasis on collaboration, NACU was able to promote the optimal uses of concrete across a diverse range of applications, including buildings, bridges, and various infrastructure projects. The founding members of NACU successfully established a platform that was the catalyst for subsequent code development and other modern structural concrete design principles and standards.

The turn of the 20th century witnessed a surge in construction activities in the United States, necessitating the need for standardized practices in concrete construction. Engineers and builders faced challenges related to material quality, structural design, and construction methods. Responding to this need, the first significant milestone in establishing concrete design standards was the publication of the inaugural NACU code, "*Standard Building Regulations for the Use of Reinforced Concrete*," in 1910. This was a concise 14-page document capturing practices of material proportioning known at the time, reinforcement, and structural behavior (ACI Committee 318, 2014).

Continual advancements in material science and structural engineering promoted a deeper understanding of concrete's properties. NACU, recognizing the significance of these advancements, played a critical role in codifying data collection from these findings. As research persisted, the provisions within the codes became increasingly intricate and tailored to specific applications.

As concrete use expanded in bridge work, infrastructure, and multistory buildings, limitations of early empirical and simplified methods became evident. In 1913, NACU transformed into the American Concrete Institute (ACI). This transformation reinforced the institute as the governing body of concrete design by expanding its scope beyond the original objectives established by NACU. The ACI continued to advance concrete technology and

construction practices, expanding its influence as the central authority throughout the industry (American Concrete Institute, 2019).

Today, the ACI 318 Structural Concrete Building Code is a testament to the institute's dedication to refining and enhancing industry standards. The ACI 318 is a comprehensive guide for professionals in structural concrete design and construction. Different committees have compiled various resources to address the complexities of specific challenges encountered throughout all design forms. This code has continually evolved, incorporating the latest research findings, technological advancements, and industry best practices, ensuring concrete structures' continual improvement and safety is in the modern era. From its early days, ACI and its predecessor emphasized committee-based, consensus-driven standards informed by both laboratory investigation and feedback from real-world construction practices (American Concrete Institute, n.d.).

2.1.2 Evolution of ACI 318: From Working Stress to Ultimate Strength

Over time, the initial NACU and NACU-derived reckoning of reinforced concrete matured into what is now known as the ACI 318 Building Code Requirements for Structural Concrete and Commentary (American Concrete Institute, 2022).

During the 1920s, an exciting period in structural engineering, the ACI, in collaboration with the Committee on Engineering Practice of the Concrete Reinforcing Steel Institute, introduced a new code titled "*Tentative Building Regulations for the Use of Reinforced Concrete*." This flagship development marked a crucial shift in focus of construction practices, particularly in using reinforced concrete—an innovative technique involving incorporating steel reinforcement within concrete to augment its tensile strength.

Reinforced concrete emerged as a groundbreaking concept in structural engineering during this period, ushering a new era in construction due to its enhanced structural integrity and design versatility. Recognizing its immense potential, the ACI redirected its focus toward formulating guidelines tailored explicitly for designing and implementing reinforced concrete structures.

To consolidate and refine the evolving standards in concrete construction, the ACI merged the aforementioned "*Tentative Building Regulations for the Use of Reinforced Concrete*" and its preceding documents. This strategic integration allowed the institute to emphasize and further develop guidelines and methodologies centered around incorporating and efficiently using reinforcing materials within concrete, cementing its commitment to advancing and standardizing the practices related to reinforced concrete construction.

The release of the "*Building Regulations for Reinforced Concrete* (ACI 318-41)" in 1941 was a significant milestone in the development of these standards. This publication laid the groundwork for the modern format and structure of the ACI 318 series, which signified another fundamental step forward in codifying and refining regulations governing the use of reinforced concrete in construction (ACI Committee 318, 2014).

The ACI 318 Committee is a cornerstone within the American Concrete Institute and pivotal in the evolution of structural concrete standards. Its genesis dates back to the mid-20th century, and its successive editions have emerged as among the most universally embraced standards governing structural concrete design and construction practices related to structural concrete, both within the United States and globally.

Subsequent editions of the ACI 318 series have undergone substantial revisions, incorporating the latest advancements in material science, engineering principles, and

construction methodologies. The ongoing evolution of these standards has solidified the ACI 318 series as a preeminent and extensively utilized guide for professionals involved in designing, analyzing, and implementing structural concrete elements in various construction projects.

Attention to detail, continual updates, and comprehensive coverage of design provisions within the ACI 318 series have established it as a cornerstone document. It provides foundational guidance and sets high benchmarks for safety, reliability, and innovation in structural concrete design and construction practices.

The year 1956 marked the next major pivot in the evolution of structural concrete standards, with the ACI 318 Committee introducing a significant shift in how the code addressed contemporary research (American Concrete Institute, 2019). Notably, within the appendix of ACI 318-56, a landmark transformation took place and it became the first documented code to integrate ultimate strength design principles for reinforced concrete members. This inclusion was a pivotal moment, setting a precedent for subsequent editions by establishing a framework that evolved to confront new challenges, assimilate advancements in research, and adapt to changing construction practices.

ACI 318-56 laid the foundation for subsequent editions, instituting a structure that adapted over time to accommodate the evolving construction landscape. It addressed critical aspects related to reinforcing steel and offered guidelines for its selection and placement. Notably, the code acknowledged diagonal tension cracking near minimal or zero moment points and presented provisions to tackle such issues.

This code edition also introduced more sophisticated structural analysis methods tailored for reinforced concrete structures. It encompassed provisions for calculating crucial parameters such as moments, shears, and other significant forces, considering the interplay between concrete

and reinforcing steel. The code established comprehensive design provisions for various structural elements, including beams, columns, and slabs, featuring formulas and guidelines to determine the requisite dimensions and reinforcement, encompassing cross-sectional analysis of different members.

ACI 318-56 also addressed aspects related to construction practices, incorporating provisions regarding formwork, curing methodologies, and quality control measures. These directives ensured that concrete structures were constructed according to specified design requirements and industry best practices.

An integral aspect integrated into this edition was the introduction of design safety factors, often referred to as "capacity reduction factors." These factors were instrumental in accounting for uncertainties inherent in material properties, construction methodologies, and applied loadings. Incorporating these design safety factors was crucial in upholding concrete structures' reliability and safety standards, ensuring they met robust safety margins despite potential variations and uncertainties in the construction process.

The evolution of the concrete design code has been marked by an increasing importance on comprehending concrete as a nonhomogeneous material and its interaction with contemporary design forces. This emphasized optimizing safety, particularly in understanding the intricate interaction between reinforcing steel and concrete.

Another notable transformation occurred in 1976 when the code underwent a significant change. Plain concrete design was eliminated from the code, prompting the name change to "*Building Code Requirements for Reinforced Concrete*." This nomenclature remained unaltered until 1989 when the code was rebranded as "*Building Code Requirements for Structural Concrete.*" Subsequently, the plain concrete design was reintroduced as chapter 22 (chapter 14

in ACI 318-19) (Hoffman, Gustafson, & Gouwens, 1998). This reintroduction was a crucial reminder of ACI's commitment to consistently update and modify concrete design practices through extensive laboratory research and testing.

A compelling aspect of this research involves exploring ancient concrete designs that have withstood the test of time. Examples such as the Ancient Roman ports, water transport systems, and structures like the Pantheon serve as fascinating case studies. Additionally, modern designs, including unreinforced archways exhibiting no signs of cracking or failure, have piqued interest in understanding their durability over time. A correlation observed in these unreinforced structures is the absence of accelerated crack propagation. The exact reasoning for that type of reaction is beyond the topic of this report, however it goes along with the idea of utilizing plain concrete in contemporary practice.

Considering the success of the aforementioned projects spurs a curiosity into the exploration of plain concrete design, focusing specifically on the possibility of its use as a carbon reduction strategy to achieve global emission reduction goals by 2050. Plain concrete structures' positive attributes and durability, especially when considering their potential role in mitigating carbon footprints and seeking sustainable construction methods, align with ambitious carbon reduction objectives. Leveraging plain concrete design as a critical component in achieving environmental sustainability goals as explored in this report represents a possible avenue for future innovations and advancements in the construction industry.

2.2 Formwork Design and Construction

The process of placing formwork involves several steps to ensure the proper alignment, shape, and dimensions of the concrete element being constructed. An overview of the placement, setting, and removal of concrete formwork is explained by the following sections.

2.2.1 Overview and Function

Foundation formwork is a temporary structural system used to shape and support freshly placed concrete until it achieves sufficient strength to independently support imposed construction loads. Proper formwork design and installation are critical to ensuring dimensional accuracy, structural performance, and surface quality of concrete elements (ACI Committee 347, 2014). The purpose of formwork is threefold:

1. To define the geometry of the concrete element in accordance with design documents.
2. To withstand the hydrostatic pressure and vibration forces exerted by fresh concrete during placement.
3. To maintain alignment, stability and surface integrity until the concrete achieves the required strength to remove the forms, usually 70-75% of the design compressive strength (Mehta & Monteiro, 2014).

In contrast to elevated structural elements, foundation formwork often benefits from soil support, which reduces design complexity and material requirements. However, improper installation or premature removal can lead to deformation, misalignment, or structural distress. Therefore, the design, placement, and removal of foundation formwork require careful coordination among the design engineer, contractor, and inspector (American Concrete Institute, 2022).

2.2.2 Design Responsibility and Documentation

Foundation formwork is typically designed by the contractor when it involves simple, soil-supported geometries such as isolated or continuous footings. For more complex systems, such as stepped or sloped foundations, deep excavations, or areas subject to lateral soil pressure,

a specialty formwork engineer may be retained to ensure adequate bracing and stability (ACI Committee 347, 2014).

Designs are prepared based on construction documents and must conform to the dimensions, elevations, and tolerances shown on the foundation drawings. These details dictate the formwork geometry, concrete cover, and required embedment locations for anchor bolts, reinforcing bars, and sleeves. Formwork design must account for:

- hydrostatic pressure from fresh concrete
- self-weight and construction live loads
- soil and groundwater pressure (for deep excavations)
- temperature and shrinkage effects
- lateral stability and alignment

All formwork for foundations must meet ACI 347, *Guide to Formwork for Concrete*, and relevant OSHA safety provisions for temporary works.

2.2.3 Site Preparation and Layout

Before installing foundation formwork, the construction site must be prepared to provide a stable base. The subgrade is graded, compacted, and leveled according to the specified bearing elevation. Survey crews transfer layout information from drawings to the field using stakes, string lines, and benchmarks, ensuring correct footing size, location, and orientation (American Concrete Institute, 2022).

For shallow foundations, earth forming, otherwise known as casting concrete directly against excavated soil, is often permitted under ACI 318 provisions, provided the soil is stable, non-erodible, and free from organic matter or water seepage. In these cases, formwork is only

required along exposed footing edges or where tolerances demand precision. In contrast, full formwork systems are necessary when:

- excavation sides are irregular or unstable
- high-quality surface finish or dimensional control is required
- reinforcement congestion prevents soil support

  water intrusion or soft subgrade conditions exist

2.2.4 Formwork Materials and Configuration

The choice of formwork material for foundations depends on reuse frequency, availability, and required finish quality. Common materials include:

- Timber and plywood: Economical and easily adaptable for isolated footings and grade beams.
- Steel panels: Durable, provide excellent dimensional control, and are used for repetitive or large-scale foundations.
- Aluminum or composite systems: Lightweight and corrosion-resistant, suitable for rapid setup in confined areas.

Form panels are secured using stakes, walers, and spreaders to maintain width and prevent bulging under lateral pressure. For wide footings, internal bracing or cross-ties may be required to resist hydraulic forces during placement.

2.2.5 Assembly, Alignment, and Bracing

After layout, formwork components are assembled on-site following the marked lines. Alignment is verified against both horizontal and vertical control points to ensure proper footing elevation and orientation.

- edge forms define the perimeter and thickness of the footing;

- kickers or nailers may be installed along the base to ensure proper anchorage; and bracing systems including struts, props, and stakes stabilize the formwork during concrete placement.

2.2.6 Application of Release Agents

To facilitate clean stripping and prevent concrete adhesion, release agents (form oils) are applied to the interior surfaces before placement. These agents also protect wood and metal forms from moisture absorption and corrosion (Kibert, 2022). Common release agents include:

- Mineral oil-based: Effective and economical for timber forms.
- Vegetable oil-based: Environmentally friendly alternative derived from renewable resources.
- Wax- or silicone-based: Produces high-quality finishes and reduces surface blemishes.

Release agents must be applied uniformly in thin coats. Overapplication can compromise surface finish or interfere with subsequent coatings or waterproofing.

2.2.7 Concrete Placement and Curing

Once formwork passes inspection, concrete is placed and consolidated within the form. For shallow foundations, direct discharge from the chute or pump is common; however, vibration is required to eliminate air pockets and ensure uniform density.

The concrete is left to cure within the formwork until it reaches sufficient strength to resist placement and construction loads, typically at least 70–75% of the design compressive strength (Mehta & Monteiro, 2014). The design engineer specifies the minimum strength and curing duration for form removal.

2.2.8 Form Removal and Backfilling

Once the removal strength is achieved, formwork is stripped carefully to prevent chipping or cracking along footing edges. Immediately after removal, backfilling or lateral restraint should be applied to prevent edge lifting or soil collapse.

Form panels should be cleaned, repaired, and stored for reuse. Proper maintenance and correct application of release agents extend service life and improve the economic efficiency of the formwork system (ACI Committee 347, 2014).

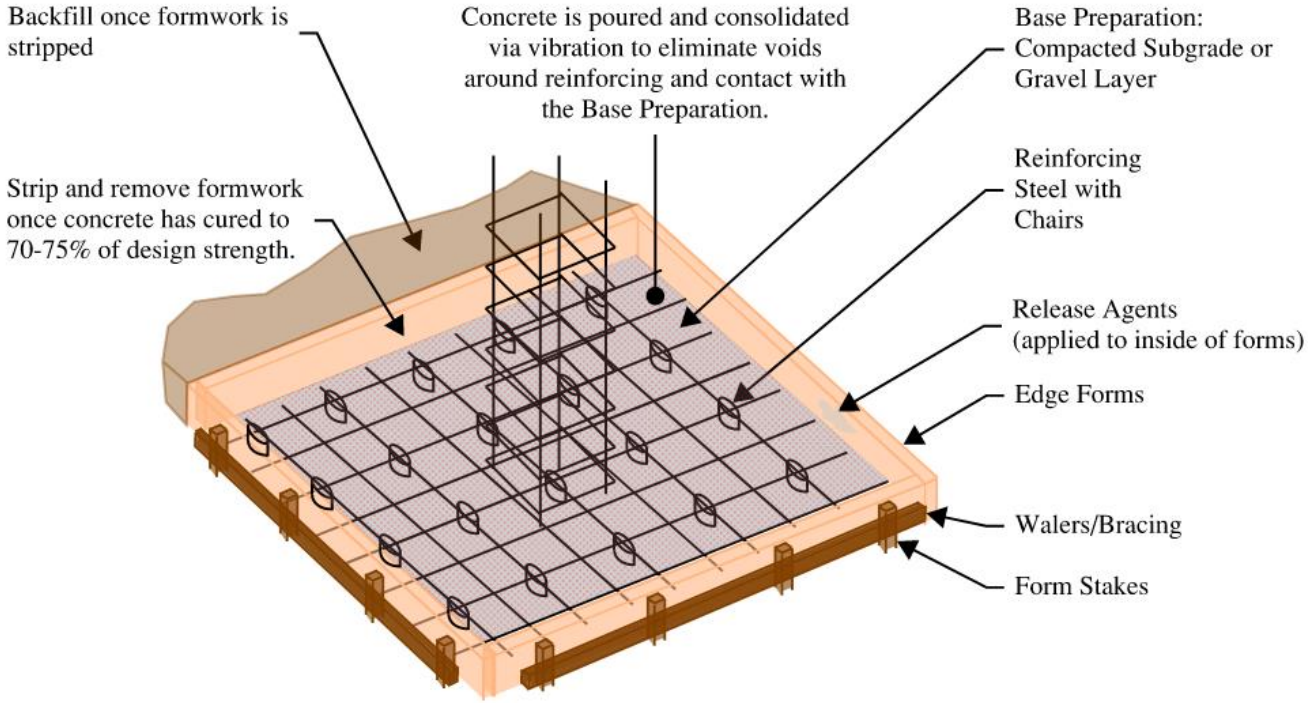


Figure 2-1 - Illustration of Typical Foundation Formwork Configuration for an Isolated Spread Footing

2.3 Regional Design Considerations and Impacts on Material Acquisition

While larger construction projects may involve the use of aggregates from quarries and mines, there is a focus on local sourcing of materials within concrete design due to the ease at which most items can be gathered on small local scales. Doing so provides time and financial

benefits to a project, and more specifically is an intelligent approach to environmentally conscious design practices.

2.3.1 Historical Context of Aggregate Sourcing in the Midwest

In the early development of construction practices within the Midwestern United States, aggregate sourcing was primarily a local endeavor. Builders relied heavily on nearby natural deposits, such as riverbeds, glacial outwash plains, and shallow quarries for sand, gravel, and crushed stone used in early concrete mixtures (National Stone, Sand & Gravel Association, 2022). These early operations employed manual extraction methods, including hand tools and animal-drawn carts, reflecting the limited mechanization of the 19th century (Mindess, Young, & Darwin, 2003).

The advent of industrialization in the late 19th and early 20th centuries profoundly transformed the aggregate supply chain. The expansion of railway systems, and later the development of interstate highways, allowed for the efficient transport of bulk materials from distant quarries to construction sites, greatly broadening the geographic range of supply (Mehta & Monteiro, 2014). This logistical advancement supported the rise of large-scale civil and structural projects, including bridges, dams, and urban infrastructure throughout the Midwest.

2.3.2 Evolution of Extraction and Processing Technologies

By the mid-20th century, demand for construction materials had grown significantly due to rapid urbanization and population expansion. To meet this demand, mechanized quarrying techniques were adopted, including drilling, blasting, and the use of conveyor systems and crushers (Kosmatka, Kerkhoff, & Panarese, 2002). The introduction of advanced screening and washing systems enabled the production of well-graded aggregates suitable for high-performance concrete applications.

Modern aggregate processing plants now employ computerized control systems to optimize crushing sequences, reduce waste, and ensure compliance with gradation requirements outlined in ASTM C33, *Standard Specification for Concrete Aggregates* (ASTM International, 2020). These innovations have not only improved production efficiency but also enhanced the consistency and durability of concrete used in structural applications.

2.3.3 Environmental and Regulatory Considerations

From the late 20th century onward, environmental stewardship has become a central focus of aggregate acquisition and production. State and federal regulations, such as those enforced by the U.S. Environmental Protection Agency (EPA) and state departments of natural resources mandate land reclamation, dust control, water management, and habitat protection (U.S. Environmental Protection Agency, 2023). Quarry operations are now subject to environmental impact assessments (EIA) and reclamation plans, which ensure that extraction activities are conducted responsibly and that disturbed land is restored for future use (U.S. Geological Survey, 2021).

Additionally, increasing value has been placed on reducing the embodied energy and carbon footprint of aggregate production. Innovations such as electric-powered crushers, conveyor belt optimization, and closed-loop water recycling have helped minimize the environmental impacts traditionally associated with aggregate processing (Pomponi, 2018).

2.3.4 Sustainable Practices and Local Sourcing

In recent decades, the construction industry has increasingly prioritized sustainable material sourcing. This shift aligns with broader goals to reduce greenhouse gas emissions, conserve natural resources, and promote circular economic principles (Kibert, 2022). Efforts toward sustainability include:

- Recycled aggregates: The reuse of crushed concrete and masonry debris from demolition projects, conforming to ASTM C856 standards.
- Industrial byproducts: Utilization of slag, fly ash, and reclaimed asphalt pavement (RAP) as supplementary materials.
- Local sourcing: Selection of nearby quarries and sand deposits to reduce transportation-related emissions and support regional economies.

  In the Midwestern region, locally available materials typically include:

  - limestone (extensively quarried in Kansas, Missouri, and Iowa);
  - dolomite and granite (from northern Missouri and southern Minnesota);
  - glacial sands and gravels (common throughout the Great Plains); and
  - recycled concrete aggregates (RCA) from regional demolition and reconstruction projects (U.S. Geological Survey, 2021).

These materials are transported via rail, truck, or inland waterways, forming a multimodal logistics network that supports cost-effective and reliable delivery to construction sites.

2.3.5 Future Directions

Looking forward, the intersection of regional design practices and material acquisition strategies will increasingly emphasize low-carbon and resilient infrastructure. Continued research into carbon capture in aggregate production, alternative binders, and sustainable quarry management will shape how the Midwest and the broader U.S. source and utilize materials for concrete construction (World Green Building Council [WGBC], 2023).

By integrating sustainability considerations into material procurement and regional design, engineers can reduce both the environmental footprint and life-cycle cost of concrete

structures, aligning with the objectives outlined in the *ACI 130: Sustainability of Concrete* committee reports (ACI Committee 130, 2021).

2.4 Construction Equipment Impact

Construction equipment plays a vital role in modern building and infrastructure projects but also represents a significant source of greenhouse gas (GHG) emissions and air pollutants globally. The majority of heavy equipment used in construction, such as excavators, bulldozers, cranes, loaders, and haul trucks are powered by internal combustion engines using fossil fuels, primarily diesel. The combustion of these fuels releases carbon dioxide (CO_2), nitrogen oxides (NO_x), and particulate matter (PM), all of which contribute to climate change, air quality degradation, and occupational exposure risks (U.S. Environmental Protection Agency, 2023); (United Nations Environment Programme, 2020).

According to the International Energy Agency (International Energy Agency, 2022), construction equipment accounts for approximately 10% of total energy-related CO_2 emissions within the building and infrastructure sector. The environmental effects of such operations extend beyond direct emissions to include indirect impacts such as fuel extraction, refining, transportation, and equipment maintenance. In an effort to reduce their impact, minimizing the time operating on site, even if by moments, adds small, compoundable change in a project's total emissions.

2.4.1 Emission Sources and Equipment Characteristics

Traditional construction machinery utilizes diesel engines due to their high torque output and energy density, which are well-suited for earthmoving and lifting operations. However, diesel combustion produces CO_2 and black carbon—short-lived climate pollutants that have a

disproportionately large warming potential compared to CO_2 alone (U.S. Environmental Protection Agency, 2023).

Typical emission factors for off-road diesel equipment range from 2.6 to 3.0 kg CO_2 per liter of diesel burned, depending on efficiency and load conditions (U.S. Environmental Protection Agency, 2023); (U.S. Department of Energy, 2022). As a result, extended equipment operation across multiple phases of a project can substantially increase the embodied carbon associated with the construction process. Key contributors include:

- excavators and bulldozers during site preparation and grading;
- cranes and lifts for structural assembly; and
- concrete pumps and mixers for material placement.

Beyond direct emissions, the transportation and maintenance of such equipment also contribute indirectly to environmental impact through additional vehicle trips and resource consumption.

2.4.2 Sustainable Equipment Practices and Technological Advancements

To mitigate the environmental burden of construction machinery, the industry has increasingly explored low-emission and alternative-fuel technologies. Emerging strategies include:

- Biodiesel and Renewable Diesel: Compatible with existing diesel engines and capable of reducing CO_2 emissions by up to 60% compared to conventional fuels (National Renewable Energy Laboratory, 2022).
- Compressed Natural Gas (CNG) and Hydrogen: Offer cleaner combustion with reduced particulate emissions, though adoption is limited by refueling infrastructure.

- Electric and Hybrid Equipment: Battery-electric excavators, loaders, and cranes eliminate on-site tailpipe emissions and significantly reduce operational noise, improving worker safety and environmental compliance (International Energy Agency, 2022).

In addition, digital technologies such as telematics, GPS-based fleet management, and predictive maintenance systems enable contractors to optimize fuel use, reduce idle time, and improve scheduling efficiency, resulting in lower total emissions and cost savings (Bo & Mjosund, 2022).

2.4.3 Economic and Operational Constraints

Despite technological progress, adoption barriers persist. Many contractors retain existing fleets rather than investing in low-emission alternatives due to capital cost, limited availability, and uncertain return on investment (World Green Building Council [WGBC], 2023). Consequently, operational decisions often prioritize time, cost, and efficiency over environmental benefits, even though these factors are intrinsically interconnected.

From a project management perspective, any inefficiencies in machinery use, such as idling, maintenance downtime, or delayed mobilization directly increase both fiscal costs and carbon emissions. For instance:

- Transport logistics require additional truck trips to deliver and remove equipment from sites, generating further CO_2 emissions before work even begins.
- Site conditions such as wet soil or inclement weather can delay heavy equipment operations, increasing total fuel use and project duration.
- Mechanical breakdowns and maintenance activities extend operation schedules, compounding emissions and resource use (U.S. Department of Energy, 2022).

Each phase of equipment mobilization, operation, and demobilization contributes cumulatively to the embodied carbon of the completed structure (Pomponi, 2018).

2.4.4 Integrated Planning and Emission Reduction Strategies

Reducing emissions associated with construction equipment requires holistic planning across the project life cycle. Key strategies include:

- Optimized Scheduling: Reducing idle time and overlapping equipment operations through digital planning tools.
- Local Sourcing and Compact Equipment Selection: Minimizing transport emissions and ensuring right-sized machinery for each task.
- Alternative Fuels and Electrification: Transitioning to low-carbon energy sources as technology and infrastructure mature.
- Lifecycle Assessment (LCA): Quantifying the embodied carbon contribution of equipment use within project-level sustainability assessments (International Organization for Standardization, 2006); (ACI Committee 130, 2021).

Minimizing unnecessary machinery use or employing manual or small-scale alternatives where feasible can yield dual benefits by reducing carbon emissions and improving schedule efficiency. Thus, environmental and economic objectives should be viewed not as competing interests, but as mutually reinforcing elements of responsible project delivery.

2.5 Factors of Safety in Structural Design

In structural engineering, factors of safety (FoS) represent a fundamental principle ensuring that designed structures maintain an adequate margin of strength and stability beyond the expected service demands. These safety provisions account for uncertainties in material

behavior, loading conditions, and construction practices to protect against potential failure (Ellingwood, Galambos, MacGregor, & Cornell, 1980).

Modern design standards, such as the *ACI 318 Building Code Requirements for Structural Concrete* and *ASCE 7 Minimum Design Loads for Buildings and Other Structures* implement safety through limit state design frameworks, where uncertainties are explicitly addressed using strength reduction factors (ϕ) and load factors (γ) (American Concrete Institute, 2022); (American Society of Civil Engineers, 2022). These approaches have replaced older "working stress" methodologies by establishing reliability-based models that align with the probability of failure at acceptable thresholds (Nowak & Collins, 2013).

2.5.1 Material Variability and Construction Tolerances

A critical consideration in safety design involves variability in material properties and construction tolerances. Structural materials such as concrete and steel are subject to inherent deviations in compressive strength, modulus of elasticity, and yield capacity due to variations in raw materials, manufacturing, and curing or fabrication processes (Neville & Brooks, Concrete technology (2nd ed.), 2010).

Similarly, construction tolerances, such as deviations in alignment, reinforcement placement, or curing conditions, introduce additional uncertainty. Although these deviations are typically within acceptable code-defined limits, their cumulative effects can alter the structural response under load. Consequently, the overall system reliability is influenced not only by individual material properties but by how these tolerances interact within the constructed assembly (Melchers & Beck, 2018).

While design codes incorporate statistical design safety factors to account for such variability, empirical research consistently indicates that in-situ material strengths often exceed

nominal design values by significant margins (Bazant & Planas, 1998). For example, concrete compressive strengths in laboratory-controlled conditions frequently test 15–30% higher than specified design strengths due to conservative assumptions and batch variability (American Concrete Institute, 2022).

2.5.2 Geotechnical Uncertainty and Comparative Safety

The use of design safety factors in geotechnical engineering offers a parallel example of conservatism in design. Reported soil bearing capacities often vary significantly between similar geologic conditions due to differences in testing procedures, local data, and interpretation of field results (Das & Sobhan, 2014). This reflects the high degree of uncertainty in subsurface conditions, due to technical difficulties in obtaining highly accurate and exact measurements across the industry.

Historical analysis of older structures, particularly those constructed prior to modern soil mechanics theory, demonstrates that many foundations perform satisfactorily despite having nominal bearing pressures that exceed contemporary code limits. This observation suggests that current safety assumptions in foundation design may be overly conservative, potentially justifying future research into improved reliability models and site-specific calibration of design parameters (Phoon & Kulhawy, 1999).

2.5.3 External Loads and Extreme Events

Design codes define prescriptive load combinations for dead, live, wind, snow, and seismic loads. However, unforeseen external or accidental events, such as impact, explosion, or extreme climatic phenomena often fall outside standard design scenarios (American Society of Civil Engineers, 2022). Engineers frequently introduce additional safety margins or adopt risk-

based design methods to account for these low-probability, high-consequence occurrences (Ellingwood, et al., 2007).

Seismic loading, in particular, presents a persistent challenge due to the difficulty of accurately replicating ground motion excitation in analytical models. Although modern codes (e.g., *ASCE 7* and *ACI 318, Chapter 18*) include sophisticated seismic provisions, the complexity and unpredictability of real seismic events mean that the true reliability of structures under extreme seismic excitation remains uncertain (Priestley, Calvi, & Kowalsky, 2007).

2.5.4 Reliability-Based Design and Statistical Interpretation

The transition from working stress design to strength-level (limit state) design in the mid-20th century marked a paradigm shift in structural safety philosophy. Under this approach, design is guided by statistical likelihood rather than deterministic safety margins, with reliability indices (β) serving as benchmarks for acceptable risk levels (Nowak & Collins, 2013).

While this probabilistic framework ensures uniform safety across structural systems, it also introduces potential over-conservatism when applied to materials or systems with well-understood performance. Continued refinement of reliability data and statistical calibration could justify higher utilization ratios for certain structural elements, particularly those where historical performance and modern quality control demonstrate consistent reliability.

2.5.5 Discussion

The continual evolution of design safety factors reflects an ongoing attempt to balance safety, economy, and sustainability in structural engineering. While conservatism remains essential to protect public welfare, excessive safety margins can increase material usage, cost, and embodied carbon, counteracting modern sustainability goals (Pomponi, 2018).

Future advancements in probabilistic modeling, real-time monitoring, and data analytics may enable dynamic adjustment of design safety factors based on actual performance data rather than purely theoretical assumptions. Such approaches could lead to optimized safety and efficiency, achieving structural reliability without unnecessary environmental or economic cost.

Chapter 3: Parametric Study on the Size and Composition of Spread Footings

3.1 Overview

A parametric study was conducted to evaluate the influence of key parameters on the performance and embodied carbon of plain concrete and reinforced concrete spread footings. The integration of embodied carbon assessment, regional material acquisition, construction equipment impact, foundation formwork design, and design safety factor evaluation establish a process-dependent approach to mitigating the environmental impact of foundation systems. The study also aims to provide a rational comparison between the two systems by quantifying the effects of design-dependent factors such as soil bearing capacity, column loading characteristics, concrete compressive strength, and footing geometry. By systematically varying parameters such as applied loading into the footing, footing geometry, material strength, and formwork configuration against an established set of environmental parameters, an array of calculations can be performed that allow for the discovery of correlations between structural safety margins, embodied carbon intensity, and construction practicality.

In terms of quantifying efficiency, the term utilization ratio, or utilization is used and accompanied by a percentage. This value is the ratio of demand over capacity, multiplied by 100 to obtain a percentage. Any ratio approaching 100% provides an efficient amount of resistance to meet the required capacity of the section. Lower percentages are both a sign of inefficiencies but also portions of design that are not as critical. Those less critical sections are discussed alongside the checks themselves. All calculations were performed in accordance with the *American Concrete Institute (ACI) 318-19(22) Building Code Requirements for Structural Concrete* (American Concrete Institute, 2022).

This parametric evaluation focuses on identifying how incremental variations in each parameter affect both the structural adequacy and environmental impact of the foundation system. The study also accounts for realistic construction conditions such as frost depth, formwork use, and material sourcing representative of the Midwestern United States. Ultimately, the adjusted variables within their defined ranges will quantify their impact on the dependent outputs: embodied carbon, material efficiency, and constructability.

3.1.1 Soil Conditions

Soil conditions represent one of the most significant sources of variability in footing design. A standard site soil-bearing capacity can be determined with the aid of geotechnical reports requested from a structural engineer. For the case of this study, the presumptive soil bearing capacity is derived from *International Building Code (IBC) Table 1806.2* (International Code Council, 2021), which provides allowable bearing values for various soil types. To represent a realistic range of conditions, three bearing capacities were selected:

- 2,000 psf: Comparable to less dense, granular soil types;
- 3,000 psf: Moderate soil densities, more stiff, cohesive soils; and
- 4,000 psf: More dense, cohesive soils.

Table 1806.2 prescribes five potential allowable bearing conditions and establishes the maximum allowed bearing pressure based on the assumed capacity of the soil type. The extreme high and low allowable bearing capacities were avoided due to their impracticality and inability to yield economical design results.

A 30-inch-deep frost depth is assumed to work for regions that experience at least modest winters, such as those in Midwestern climates. Locating the top of the footing at two feet six inches below grade prevents heaving. This is consistent with regional design practices. In

conjunction with *ACI 318-19(22) Table 19.3.2.1*, a moderate range of concrete compressive strength is specified in a later section in conjunction with this frost assumption.

Seismic characteristics for a site may also be provided through a geotechnical report. However, those variables will not be addressed in the example calculation, and it will be assumed that the site where the example occurs is in a relatively low seismic hazard region (Seismic Design Category A or B).

3.1.2 Building Framing and Column Loading

A set of arbitrary building heights is evaluated in this study, ranging between one, two, and three stories. To maintain generality, the study considers a steel-framed building with concentrically loaded columns bearing on individual spread footings. The column load is transferred through a steel base plate, creating a critical bearing area at the interface with the concrete footing. Addressing *ACI 318-19(22) Table 13.2.7.1* the effective critical area is approximated by a 16 by 16-inch square at the column base, this size allows for ample space to provide anchors and have uniform load distribution. Using the 16 by 16-inch square measurement creates a less intense loading area as compared to a 12 by 12-inch square critical area. If smaller-sized members are selected that could handle similar loading conditions to what is outlined here, marginally higher values in quantity can be expected. Based on this critical area assumption, the exact size of the column member does not impact the overall design procedure for the concrete footing.

Axial loading imposed by the column onto the spread footing assumes a 40:60 dead-to-live load ratio, typical for occupancies with moderate live load variations such as commercial or light industrial structures. This ratio enables conversion between service-level and ultimate loads, forming the basis for determining footing area and depth. Structures exceeding three

stories or experiencing significant lateral demands were excluded from this analysis due to differing foundation system requirements.

3.1.3 Concrete Strength and Type

The compressive strength of concrete, f'_c, plays a critical role in determining both the structural capacity and environmental footprint of spread footings. Concrete strengths vary based on project requirements, with the lowest acceptable specified compressive strength for a site in Seismic Categories A, B, or C is 2500 pounds per square inch per *Table 1808.8.1 of the ICC* (International Code Council, 2021). In this study, a compressive concrete strength of either 3000 or 4000 pounds per square inch has been selected due to the frequency that these strengths are typically specified in the industry for use in shallow foundation systems of low- to mid-rise structures. The *ACI 318-19(22)* has site-based requirements for a variety of environmental conditions, defined by *Table 19.3.1.1*. Based on the frost depth assumption established, a moderate frost depth of (30 inches) and minimal exposure to chlorides or sulfates, this example footing will not be exposed to freezing, and therefore these values meet the required strengths for the F0 exposure class per *Table 19.3.2.1*. Other site conditions that would warrant a minimum strength of concrete would be exposure to sulfate and corrosive materials such as chlorides, which again, is not an issue for this study. The last environmental condition not explicitly considered from *Table 19.3.1.1* is concrete being in constant contact with water. With those site assumptions, the two values of concrete compressive strength examined would pass at any site with the same conditions outlined. If more intense site conditions are experienced, then an increased concrete compressive strength will be required in accordance with *Table 19.3.2.1*. One likely condition that could be faced is increased frost exposure from class F0 to F1, where the members are now exposed to freeze and thaw cycles, but still not exposed directly to snow and

ice. If that happens, then only the 4000 pound per square inch compressive strength examples would be viable. In more severe exposure conditions where foundation elements come into contact with snow and ice, or are exposed to deicing chemicals, then conditions F2 or F3 are met. Both of the studied strengths would be inadequate for those cases. Additionally, it is important to note that F3 exposure conditions impose a limit on the compressive strength that differs between plain and reinforced concrete. It is mandated that minimum strengths of 4,500 psi for plain concrete and 5,000 psi for reinforced concrete are used. These distinctions underscore the importance of environmental classification in durability design, particularly for foundations near grade or in cold regions.

The mechanical behavior of concrete is not solely defined by its compressive strength; it also depends on aggregate composition, cementitious content, water-to-cement (w/c) ratio, and curing conditions. Normal-weight concrete is typically sourced due to the minimal required processing after harvest and therefore will be the choice of concrete type for this study. Concrete materials are readily available due to the large number of manufacturers located across the United States. For example, companies such as Geiger Ready-Mix (Geiger Ready-Mix Co., 2024) and Midwest Concrete Materials (Midwest Concrete Materials, Inc., 2024) service a large portion of the Kansas and Western Missouri region. All materials are assumed to conform to *ASTM C150/C150M* for portland cement and *ASTM C33/C33M* for concrete aggregates (ASTM International, 2020); (ASTM International, 2020). Locally sourced aggregates reduce transport-related embodied carbon while maintaining compliance with *ACI 301-20: Specifications for Concrete Construction* (ACI Committee 301, 2020). Cost is also controlled through the aforementioned availability of materials. With there being no extraneous external factors such as

seismic activity or aggressive weather events, there's no need for a complicated or high-strength mix.

To ensure consistency between specimens, air entrainment is assumed to be incorporated in all mixes at a nominal 5% ± 1% by volume, consistent with *ACI 201.2R-16* (ACI Committee 201, 2016) recommendations for concretes exposed to cyclic freezing and thawing, even under moderate exposure conditions. Entrained air improves durability and resistance to scaling, particularly when footings are placed near the frost line.

In summary, this study assumes a moderate-strength, normal-weight concrete with a typical w/c ratio of 0.45–0.50, with the previously established compressive strength of either 3000 or 4000 pounds per square inch mix is specified for the set of examples later on.

3.1.4 Footing Dimensions and Formwork Considerations

The geometry of a spread footing is governed primarily by the applied column loads, soil bearing capacity, and structural strength limitations related to bending and shear. The design also incorporates practical construction constraints such as formwork configuration, concrete cover, and frost protection requirements. These interrelated factors determine the plan dimensions (width and length) and thickness (depth) of both plain and reinforced concrete footings.

This section defines the basis for selecting footing dimensions used in the parametric study and establishes the design assumptions necessary for evaluating both embodied carbon and structural performance in accordance with.

The width and length of the footing rely mainly on the loading and soil capacities as defined previously; depth, however, is specified by a combination of the *ACI 318-19(22)* code minimum dimension requirements and limitations in nominal strength resisting bending and

shears. Depth of a footing is necessary for determining the critical sections, which provide the location for where applied loads and internal stresses reach their maximum values. According to *ACI 318-19(22) Section 13.2.7.1*, the critical section for one-way shear occurs at a distance equal to the effective depth (d) from the face of the column, pedestal, or wall. For two-way (punching) shear, the critical section is located at d/2 from the periphery of the loaded area.

In this study, the column base plate and steel column combination is assumed to produce a 16 by 16-inch critical area, based on *ACI 318-19(22) Table 13.2.7.1*. Within this example, the footing thicknesses are between 12 and 18 inches, therefore one-way shear critical locations are then located at the thickness value away from the "face" of the 16 by 16-inch critical area. One-way shear areas are only measured away from one face, unlike their two-way counterpart. Two-way critical section perimeters are then located approximately 6 to 9 inches from either face of a steel column (so in total 12 to 18 inches on top of the critical area). This perimeter value around the critical section is measuring the full area of the footing.

As for the critical moment section, it also takes the area away from one face, like the one-way shear check, but that is purely measured from the extent of the critical area and not impacted by section depth. However, moment capacity is heavily influenced by the depth of the footing section.

The effective depth and corresponding shear perimeters are then used in the design calculations to verify that the concrete section, plain or reinforced, provides sufficient shear capacity (ϕV_c) and moment resistance (ϕM_n). The critical section dimensions therefore directly influence the footing's required thickness, and consequently, its material volume and embodied carbon.

The process of determining the previously mentioned example section thicknesses initially began with the code minimum thickness specified in *14.3.2.1* for plain concrete sections, which is 8 inches for plain sections. This nearly meets the requirements for reinforced sections outlined in *13.3.1.2* for shallow footings, which states that bottom reinforcing steel should be at an effective depth of at least 6 inches from the top of the section. With required clearances for reinforcing, 8 inches is not enough depth, so the new minimum for reinforced concrete sections was 9 inches. However, taking into consideration general clearances for items such as concrete anchor bolts and partially influenced by early calculation results with the minimum sections, the sizes of plain and reinforced sections were quickly iterated as described in the following paragraph.

For reinforced concrete, the overall width dimensions may change based on the inclusion or exclusion of formwork. With the inclusion of formwork, *Table 20.5.1.3.1* mandates that the minimum concrete coverage for reinforcing steel is 2 inches. Without formwork, the minimum coverage is 3 inches (American Concrete Institute, 2022). The code views this as an adequate correction to combat the unevenness of the soil that the concrete bears against, as well as providing plenty of protection against moisture to protect reinforcing steel from corrosion. Plain concrete sections do not have a width adjustment specified within the code based on the use of formwork. However, when calculating the nominal strengths of plain concrete sections, the effective depth for plain sections is assumed to be 2 inches shallower if cast against the soil per *Section 14.5.1.7*. This reduction in section height has been discussed as a method to account for the same unevenness and poor concrete setting at the interface between concrete and soil. For reinforced concrete sections, the depth of the footing is not specifically impacted due to formwork either, but it is impacted by clear cover requirements for reinforcing steel. The

effective depth (d) is determined from the extreme compression fiber to the centroid of the tension reinforcement. For this parametric study example set, the nominal footing thickness of the reinforced example sections were 12 inches. With 3-inch concrete cover from the bottom to the reinforcing steel, the effective depth is estimated at $d \approx 9$ inches, passing previous criteria. For the plain concrete example sections, considering the reduced effective depth of ($h - 2$ in.), unreduced section depths were finalized at 17 or 18 inches.

Formwork for spread footings would be comprised primarily of OSB or other cheap, low-quality woods that are readily available. Although other options are available, this is the most common and simplest to assemble generally. Omitting formwork then would affect not only construction efficiency and cost but also embodied carbon. Earth forming reduces material waste and labor while slightly increasing concrete volume due to irregular excavation geometry. The trade-off between constructability and sustainability forms an important consideration in the comparative evaluation between plain and reinforced footings.

In summary, to maintain consistency across comparative cases, the following base configuration was used for both plain and reinforced footings:

- Column baseplate area: 16 in. by 16 in. (critical loaded area)
- Effective depth (d): approximately 9 in. for reinforced; ($h - 2$ in.) for plain
- Concrete cover: 2 in. (with formwork) or 3 in. (earth-formed)
- Soil bearing capacity range: 2,000, 3,000, and 4,000 psf
- Concrete strengths ($f'c$): 3,000 psi and 4,000 psi

From these parameters, footing plan dimensions were derived by equating applied service loads to allowable soil pressures. The resulting thicknesses described were iteratively adjusted to satisfy flexural and shear demands at the critical section while maintaining compliance with

code-mandated minimums. This procedure produces a matrix of foundation configurations enabling direct comparison between plain concrete (earth-formed) and reinforced concrete (earth-formed). Each scenario varies in material volume, formwork requirement, and reinforcement content. Overall dimensions will then be used to quantify the embodied carbon associated with either plain or concrete designs.

3.1.5 Reinforcing Steel

The inclusion of reinforcing steel represents a defining distinction between plain and reinforced concrete spread footings. Reinforcement provides the necessary tensile capacity and ductility that plain concrete lacks, allowing the section to safely resist flexural and shear stresses developed under applied loads. This section establishes the material properties, design basis, and sustainability implications for reinforcing steel used in the parametric study.

Reinforcing steel used in all scenarios will be standard Grade 60 deformed bars, with a minimum specified yield strength, f_y, of 60,000 pounds per square inch. The modulus of elasticity is approximately 29,000,000 pounds per square inch and align with *ASTM A615/A615M-20: Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement* and are consistent with requirements in *ACI 318-19(22) Chapter 20* (American Concrete Institute, 2022); (ASTM International, 2020).

The use of deformed bars assists with the bond performance between steel and concrete. The ribs on the reinforcing bar interlock with the setting concrete to produce this interaction. All bars are assumed to meet *ASTM A706/A706M-21* (ASTM International, 2021) chemical composition and ductility limits, providing sufficient elongation for redistribution of internal stresses prior to failure.

Bar size, spacing, and layout is determined through calculations referencing *ACI 318-19(22) Chapter 15* to ensure adequate minimum flexural strength and shear resistance requirements for isolated footings. Minimum reinforcing requirements also must abide by *ACI 318-19(22) Section 14.4.2.2* which specifies the lowest amount of steel required to prevent sudden cracking and maintain ductile section behavior. Bar spacing relies on the required minimums stated by incorporating them into an iterative design process that yields the least amount of bars required for adequate section strength. Spacing utilizes uniform, whole integer measurements that reduce complexity in the construction process. This uniform distribution of bars in orthogonal directions resist bending moments in both axes. To do so, a single layer of bars near the bottom of the footing is used that is consistent with *ACI 318-19(22) Section 13.3.1.2*. Reinforcing steel inclusion will follow the absolute minimum necessity to achieve adequate strength design requirements.

The inclusion of reinforcing steel in design will impact the overall section width and depth which has a direct impact on the volume of concrete used. This will mark a major difference point between the sustainability of reinforced concrete designs versus the plain concrete design counterparts.

3.2 Plain Concrete Design of Isolated Shallow Footing Example

A series of six example calculations were conducted to evaluate and compare the behavior of plain and reinforced concrete spread footings under identical loading and site conditions. Each set of calculations follows the same controlled parameters, such as soil bearing capacity, critical section geometry, and concrete compressive strength to isolate the influence of reinforcement on footing performance.

Three of these examples represent plain concrete footings, while the remaining three represent reinforced concrete footings. Both sets are analyzed under equivalent design assumptions to allow direct comparison of required plan areas, footing depths, and capacity utilizations.

This section presents the plain concrete footing design (Examples 1 through 3). The subsequent section, Section 3.3, will outline the reinforced concrete footing analysis.

3.2.1 Design Overview and Input Parameters

The example configuration assumes the aforementioned 16 by 16-inch square loaded area, corresponding to a steel column baseplate concentrically transferring an ultimate axial load of 100 kips to the footing. Each footing is designed using normal-weight concrete with compressive strengths (f'_c) of 3,000 psi or 4,000 psi, and soil bearing capacities (q) ranging between 2,000 and 4,000 psf.

To establish comparative conditions, three representative cases are defined within Table 3-1:

Example	Soil Bearing Capacity (psf)	Concrete Strength (psi)	Environmental Severity	Design Category
1	2,000	3,000	Worst-case (soft soil, low strength)	Baseline comparison
2	3,000	4,000	Moderate soil, high strength	Typical design
3	4,000	3,000	Dense soil, moderate strength	High-support condition

Table 3-1 Parametric Study Examples – Site and Material Definitions

All design procedures conform to *ACI 318-19(22) Chapter 14: Plain Concrete Design* and *Chapter 13: Footings* (American Concrete Institute, 2022). The summarized results of this highlighted example of the parametric study are found within Appendix C.9 of this report.

3.2.2 Determination of Footing Plan Area

The first step in the footing design process is to determine the required plan area based on the unfactored service load and the allowable soil bearing capacity. Given an ultimate axial load P_u = 100 kips, the dead-to-live load ratio is assumed as 40:60, which is typical for building gravity systems supporting both self-weight and occupancy live loads. This ratio produces an unfactored service load of approximately 69.5 kips.

$$A = \frac{P}{q} \qquad \textbf{Equation 3.2-1}$$

Where:

A = Required footing area (ft^2)

P = Unfactored axial load (kips)

q = Allowable soil bearing capacity (psf)

The calculated areas for each case are 34.8 ft², 23.2 ft², and 17.4 ft², corresponding to required square footing dimensions of approximately 6.0 ft, 5.0 ft, and 4.5 ft, respectively. For constructability and standardization, each dimension is rounded up to the nearest 6 in increment, resulting in 6.0 ft by 6.0 ft, 5.0 ft by 5.0 ft, and 4.5 ft by 4.5 ft footings. These revised dimensions are subsequently used to compute factored soil pressures and ultimate moment demands.

3.2.3 Factored Soil Pressure and Ultimate Moment

The factored soil pressure (q_u) is determined using the factored axial load divided by the design area:

$$q_u = \frac{P_u}{A}$$ **Equation 3.2-2**

Resulting in factored soil pressures of 2.78, 4.00, and 4.94 ksf for the three cases.

The ultimate bending moment is derived based on a concentrically loaded column and the critical section geometry defined by *ACI 318-19(22) Section 13.2.7.1*, which locates the bending critical section halfway between the column face and edge of the baseplate. The governing relationship is expressed as:

$$M_u = \frac{q_u \left(\frac{L-N}{2}\right)^2 (B)}{2}$$ **Equation 3.2-3**

Where:

M_u = Ultimate bending moment (kip-ft)

q_u = Factored soil bearing pressure, (ksf)

$L = B$ = Footing length and width (ft)

N = Loaded area width (ft)

Substituting the parameters above yields ultimate moments of 45.4, 33.6, and 27.9 kip-ft for Examples 1–3, respectively.

3.2.4 Nominal Moment Capacity of Plain Concrete

The nominal flexural strength of plain concrete is computed using *ACI 318-19(22) Section 14.5.2.1*, which defines the limit states for tensile cracking and compressive crushing. The lesser of Equations (14.5.2.1a) and (14.5.2.1b) governs the design moment capacity:

$$M_n = 5\lambda\sqrt{f'_c}S_m$$ **Equation 3.2-4**

and

$$M_n = 0.85f'_cS_m$$ **Equation 3.2-5**

Where:

λ = Concrete type modification factor

S_m = Elastic section modulus (in^3)

The section modulus for a plain rectangular section cast directly against soil is determined as:

$$S_m = \frac{B(h-2)^2}{6}$$ **Equation 3.2-6**

Where:

h = Total footing thickness (in)

The 2-inch reduction accounts for the depth adjustment required by *ACI 318-19(22) Section 14.5.1.7* for earth-formed members.

Through iterative calculations, the required footing depths to achieve unity-level moment utilization are found to be 19 in, 17 in, and 18 in, yielding section moduli of 3,468, 2,250, and 2,304 in^3, respectively.

Applying a strength reduction factor (ϕ =0.6) for plain concrete, the design moment capacities are calculated as 47.5, 36.5, and 31.5 kip-ft, each closely matching their corresponding demand moments M_u. This demonstrates that bending is the governing design limit state for plain footings, confirming the material's low tensile capacity and limited ductility in flexure.

3.2.5 Shear and Bearing Checks

3.2.5.1 One-Way and Two-Way Shear

Ultimate shear forces are evaluated at the critical sections defined in *ACI 318-19(22) Sections 13.2.7.2–13.2.7.3*.

The governing expressions for one-way and two-way shear are:

$$V_u = \left(\frac{L}{2} - \frac{N}{2} - h\right) B q_u$$ **Equation 3.2-7**

and

$$V_u = q_u\big((BL) - (h+N)^2\big) \qquad \textbf{Equation 3.2-8}$$

Calculated shear demands are 12.5 kips (one-way) and 76.4 kips (two-way) for Example 1, reducing proportionally with the higher soil capacities in Examples 2 and 3. Comparison with code-based nominal shear capacities confirms that shear strength exceeds bending demands for all cases, consistent with the expected behavior of plain concrete footings.

3.2.5.2 Bearing Strength

Concrete bearing is evaluated using *ACI 318-19(22) Section 14.5.3*, which defines allowable bearing stress at the interface of column and footing. The critical bearing area extends outward at a 1:2 slope from the loaded area.

Bearing capacities for the three footings were computed as 783.4, 1,044.5, and 783.4 kips, indicating utilization ratios well below unity. This verifies that bearing rarely governs in isolated spread footings under moderate building loads, and bending remains the controlling limit state for plain concrete foundations.

3.2.6 Discussion

The results illustrate that plain concrete footings, although structurally feasible under moderate loading, require substantially greater depth to resist bending compared to reinforced designs. While the omission of reinforcing steel reduces embodied carbon and construction complexity, it simultaneously increases concrete volume, excavation depth, and curing time. From a sustainability perspective, the trade-off between material volume and reinforcement intensity is a key factor later quantified in the parametric analysis.

These findings reinforce the modern engineering rationale for incorporating reinforcement to improve flexural performance while balancing environmental efficiency through optimized geometry and material selection.

3.3 Reinforced Concrete Design of Isolated Shallow Footing

As outlined in the preceding section, the design parameters for the reinforced concrete spread footing examples are intentionally consistent with those used in the plain concrete analysis. This allows for a direct comparison of behavior, strength, and material efficiency between the two design methodologies under identical load conditions.

Note, for this section the incorporation of a development length of reinforcing check is conducted to ensure adequate section design per the requirements of *Section 13.2.8*. This does adjust the design base and length measurements of footings for the sake of detailing. Primarily, this adjustment occurs in lower loaded sections and increases in likelihood for sections placed with favorable soil conditions – specifically here the 4,000 psf sections. Increasing the soil's ability to resist loads demands a smaller section per Equation 3.2-1, but this causes concern for minimum requirements from development length. Increasing the volume of reinforced sections to accommodate the development length has a negative impact on optimizing the embodied carbon of these footings.

3.3.1 Design Parameters and Dimensional Adjustments

The reinforced footing examples maintain the same plan areas as the corresponding plain footings, with an additional three inches of concrete added to each side. This adjustment accommodates the code-required concrete cover for reinforcing steel when the footing is cast directly against soil, in accordance with *ACI 318-19(22) Table 20.5.1.3.1*. Additionally, the development length of reinforcing steel is also checked, and may influence the overall dimension

requirements of the footing. Within these examples, the footing with the best soil bearing conditions has an increased section base and width due to development length requirements. The resulting footing plan dimensions are 6.5 ft, 5.5 ft, and 5.5 ft, respectively. A uniform footing depth of 12 inches was selected for all reinforced examples to ensure consistency in comparative evaluation.

3.3.2 Reinforcing Steel Design and Layout

The design of flexural reinforcement follows the fundamental equations presented in *Design of Reinforced Concrete* by J.C. McCormac and R.H. Brown (McCormac & Brown, 2016), which have been modified slightly for application to spread footings. These equations are used to determine the required area of steel (A_s) based on the applied ultimate moment, section geometry, and material strengths. From this procedure, the required reinforcement areas were determined as 2.113 in^2, 1.788 in^2, and 1.625 in^2, respectively.

Per *ACI 318-19(22) Section 7.7.2.4*, the minimum spacing for reinforcing bars in footings is the lesser of five times the member thickness ($5h$) or 18 inches.
For a 12-inch-deep footing, the flat 18-inch limit governs.

The total reinforcing bar length ($L_{reinf.}$) is then calculated using Equation 3.3-1:

$$L_{reinf.} = (B - 2 * Cover)\left(\frac{B - 2 * Cover}{S_{min}} + 1\right) \qquad \textbf{Equation 3.3-1}$$

Where:

B = Base length of the footing (ft)

Cover = Concrete cover per *ACI 318-19 Table 20.5.1.3.1*

S_{min} = Minimum bar spacing (in)

Since all sides are cast against soil, cover is subtracted from both ends. Equation 3.3-1 provides in the first set of parentheses the statement of footing base length minus concrete cover

multiplied by 2. This portion is the exact length of a bar in the direction '*B*', and since this is a square footing, it works in either direction. This value accommodates the possibly updated values impacted by the development length check performed. The other portion of the equation is that same length equation, now divided by the minimum bar spacing. Due to this being a square footing, the equation may look identical but is intended to consider the orthogonal direction to the bar length calculated before. The additional plus one accounts for first counted value in a total based on divided length. Now a total length of bar is accounted for in one direction and lastly must be multiplied by two when considering the total weight of reinforcing steel for embodied carbon calculations later on. Based on these parameters, the selected bar arrangements were: Five No. 6 bars, Five No. 6 bars, and Four No. 6 bars, respectively. Each No. 6 bar weighs 1.502 lb/ft, providing an efficient balance of economy and required steel area.

3.3.3 Flexural Strength and Utilization

Using the reinforcement provided and *ACI 318-19(22)* flexural design relationships, the nominal moment capacities were determined to be: 77.2 kip-ft, 77.5 kip-ft, and 61.6 kip-ft, respectively. Applying a strength reduction factor (ϕ) of 0.90 for flexure *ACI 318-19(22) Section 21.2.*1, the utilization ratios for each section are 67%, 51%, and 55%, respectively. This demonstrates that each section is conservatively proportioned, with adequate flexural resistance and a safety margin appropriate for reinforced foundation design.

3.3.4 One-Way Shear Strength

The effective depth (d) from the extreme compression fiber to the centroid of the reinforcement governs the one-way shear check.

Given:

- Total depth, h =12 in.

- Bar diameter (No. 6) = 0.75 in.
- Concrete cover (earth-cast) = 3.0 in.

The effective depth is:

$$d = h - \left(Cover + \frac{d_b}{2}\right)$$ **Equation 3.3-2**

Where:

d_b = Bar diameter (in)

The result of Equation 3.3-2 is 8.125 inches. Using *ACI 318-19(22) Section 22.5.5.1* (nonprestressed members without shear reinforcement), the one-way shear capacities were calculated as 31.5 kips, 32.6 kips, and 24.6 kips for the three examples. One-way shear remains a secondary design check, as none of the sections approach critical capacity.

3.3.5 Two-Way (Punching) Shear Strength

Two-way shear resistance is influenced by both effective depth and critical perimeter location, determined per *ACI 318-19(22) Section 22.6.4.1* at a distance *d/2* from the loaded area. The calculated two-way shear capacities are: 128.8 kips, 148.8 kips, and 128.8 kips for the example footings.

The results indicate that the two-way shear demand-to-capacity ratios are within acceptable limits, confirming that flexure remains the governing design condition for all three footings.

3.3.6 Bearing Strength

Bearing resistance, as with plain concrete, is unaffected by the inclusion of reinforcing steel. The design follows *ACI 318-19(22) Section 22.8*, using the same area-based nominal bearing equations. The calculated bearing capacities are: 848.6 kips, 1131.5 kips, and 848.6 kips respectively.

All bearing checks demonstrate ample capacity, confirming that bearing is not a governing limit state for these footing examples.

3.3.7 Summary and Comparison Notes

The reinforced concrete footing examples demonstrate the material efficiency achievable through the inclusion of reinforcing steel, with significantly reduced section depth and improved flexural performance compared to plain concrete counterparts. However, one- and two-way shear checks indicate that depth remains a key parameter in achieving code compliance and could lead to inefficient material use by an engineer, regardless of reinforcing. Bearing capacity continues to show large safety margins, reinforcing its secondary nature in shallow foundation design.

3.4 Code Analysis of the Plain Concrete Design Example

3.4.1 Scope and Governing Documents

Design of isolated spread footings follows *ACI 318-19(22) Chapter 13*, specifically *Section 13.3 – Shallow Footings*. This section is used regardless of whether the footing is plain or reinforced. *Chapter 13* establishes the footprint of checks, load paths, and critical sections. For plain concrete footings, *Chapter 13* explicitly permits design by *Chapter 14 – Plain Concrete* (American Concrete Institute, 2022). For reinforced footings, *Chapter 13* references the appropriate sectional-strength provisions in *Chapter 22* and, for slab-type behavior, the detailing/check locations summarized in *Chapters 8 (Two-Way Slabs)* as applicable, see *ACI 318-19(22) Section 13.3.3*.

3.4.2 Critical Sections and Where They Come From

Section 13.2.5.1 directs the user *to Chapter 14*, the code provision design of plain concrete members. For determining critical sections of plain concrete members, *Section 14.4.3*

contains the guidance for all force and stress conditions that a footing will experience. They are summarized as follows:

- Bending (M_u) critical section: halfway between face of column and edge of steel base plate (column with steel baseplate) per *Table 14.4.3.2.1*, within *Section 14.4.3.2*.
- One-way (beam) shear: at a distance *h* (effective depth) from the face of the supported member, *Section 14.4.3.3.1*.
- Two-way (punching) shear: along a perimeter, b_o, located h/2 from the loaded area, *Section 14.4.3.4.1*.

Note: The location logic is consistent with the shear-stress trajectories of one-way slabs described in *Section 7.4.3.2* and *Section 9.4.3.2* commentary (45° load spread) and is mirrored in *Section 8.4.4.1.1* (directly referencing *Section 22.6.4.1*) locating for two-way slab systems. *Chapter 14*'s table governs for foundations with nonconcrete supported members (e.g., steel baseplates), ensuring correct geometry for M_u and V_u evaluation. These references are nearly identical to *Chapter 13*'s method for determining critical sections, a key difference being the difference in height of section parameters for the shear sections.

3.4.3 Plain Concrete Design Path and Adjustment from Reinforced Section Design

When the footing is designed as plain concrete, *Chapter 14* supplies material limits, minimum sizes, and sectional-strength equations.

- Use limitations: Plain concrete is generally not permitted in Seismic Design Categories D/E/F, with narrow exceptions in *ACI 318-19(22) Section 14.1.4*. Selection of exposure classes and minimum f'_c still follows *Chapter 19*.

- Minimum thickness: 8 in for isolated footings per *Section 14.3.2.1*. Commentary emphasizes that flexure is the governing limit state for plain members; thickness directly drives section modulus.
- Base area: Use service (unfactored) loads to size footprint for bearing on soil per *Section 14.3.2.2*.
- Required strength checks in *Section 14.4.3* (with critical sections as stated previously in section 3.4.2 of this report):
    - factored moment per *Section 14.4.3.2*; *Table 14.4.3.2.1* gives the location for M_u;
    - factored one-way shear per *Section 14.4.3.3*; and
    - factored two-way shear per *Section 14.4.3.4.*
- Design strength, *Chapter 14.5*:
    - General per *Section 14.5.1*: linear strain distribution and rectangular stress block may be used; when cast against soil, reduce usable depth by 2 in. per *Section 14.5.1.7* to reflect surface irregularities (this is a key distinction from reinforced members).
    - Flexure per *Section 14.5.2*: nominal moment from uncracked section; M_n governed by the lesser of tension and compression expressions (e.g., Equation 3.2-4 and Equation 3.2-5), then $\phi = 0.60$ applies to plain concrete.
    - Axial compression per *Section 14.5.3* and flexure and axial compression per *Section 14.5.4* use the same conceptual stress block as reinforced design, but without steel.

- Shear per *Section 14.5.5*: nominal shear V_c based on uncracked concrete; equations are analogous to *Chapter 22* but with different coefficients and depth definitions.
- Bearing per *Section 14.5.6*: identical basis to *Chapter 22* (area-dependent), as bearing is a compressive phenomenon in concrete.

Why *Chapter 14* differs from *Chapter 22*: Plain-concrete formulas assume uncracked behavior; reinforced formulas assume cracked sections with steel carrying tension (see *R14.5.5.1* and *R22.5.1.1*). Coefficients and effective depths therefore differ slightly even where expressions appear similar.

3.4.4 Reinforced Concrete Path (Chapters 13, 7, 8, and 22)

For reinforced footings, *ACI 318-19(22) Chapter 13* sends the designer to sectional-strength provisions in *Chapters 7, 8, and 22*; critical-section locations are those of *Section 13.2.7*, which are identical to the provisions discussed for plain concrete in *Chapter 14*.

- Omission of size effect: *Section 13.2.6.2* allows ignoring size-effect factors of *Section 22.5* and *Section 22.6* for one- and two-way shear in footings.
- Development Length of Reinforcement: *Section 13.2.8* requires the slabs to update their lengths if they do not provide the minimum development length specified in *Chapter 25*.
- Uniform reinforcement: *Section 13.3.3* references slab provisions (*Chapter 8*) for uniform distribution of reinforcement in both directions and for restating critical-section locations (*Chapter 8 Sections 8.4.2–8.4.4*).
- Sectional strength, *Chapter 22*:

- General assumptions per *Section 22.2*: linear strain distribution, $\epsilon_{c.max} = 0.003$, tensile strength of concrete neglected in flexure and axial strength of reinforced members.
- Flexure per *Section 22.3* – with many references back to the general assumptions portion: equivalent rectangular stress block, $a = \beta_1 c$; ϕ per *Chapter 21* (e.g., $\phi = 0.90$ for flexure).
- Axial/combined per *Section 22.4*: use *Table 22.4.2.1* and P_o per *Section 22.4.2.2*.
- One-way shear per *Section 22.5*: $V_n = V_c$ for footings without detailed shear reinforcing steel (stirrups), as is assumed for the parametric study and further calculations. Use *Section 22.5.5* for V_c and apply limits in *Section 22.5.5.1.1*, size effect waived by *Section 13.2.6.2*.
- Two-way (punching) shear per *Section 22.6*: define critical perimeter b_o per *Section 22.6.4* at $d/2$; compute nominal shear stress and compare to v_c per *Section 22.6.5*.
- Bearing per Section 22.8: same tables and mechanics as *Chapter 14;* see *Figure R22.8.3.2* for load spread (≈ 1:2).

3.4.5 Practical Implications for the Parametric Study

- Critical sections from *Table 14.4.3.2.1* (*Table 13.2.7.1* for reinforced concrete) govern where to take M_u, V_u (one way), and V_u (two-way).
- Plain concrete requires greater thickness to satisfy flexure (uncracked assumption, $\phi = 0.60$, and the 2-in depth reduction when earth-formed), often making bending the governing limit state.

- Reinforced concrete uses cracked-section theory with steel in tension, generally leading to thinner footings for the same loads, while shear and bearing typically remain non-governing for moderate-rise buildings.
- Bearing checks are identical in basis for plain and reinforced footings (area-controlled), which is why bearing rarely governs until loads are high or loaded areas are small.
  For the steel-column-to-baseplate-to-footing case in this report, *Chapter 14*, *Table 14.4.3.2.1* is the correct reference for critical locations; *Chapter 13*, *Table 13.2.7.1* slab provisions in *Chapter 8* are supportive but secondary.

Chapter 4: Results

A comprehensive set of calculations was performed following the design parameters established in Chapter 3 to evaluate the performance and efficiency of plain and reinforced concrete isolated footings under varying loading and soil conditions. The results presented herein are organized to highlight the influence of soil bearing capacity, concrete strength, and applied loading on the overall structural efficiency, constructability, and embodied carbon impacts of each footing type. Complete data tables for all 48 design cases are provided in Appendix C - Calculation Result Tables.

An additional set of four calculations were created and examined after analysis of the 48 designs due to an indication of plain sections having a noticeable improvement in all three results categories against their reinforced counterparts. This is explored in Section 4.7 Comparative Summary and Discussion of this chapter, and their results are found in the same appendix.

4.1 Study Overview

The parametric study focused on three principal variables that govern footing performance:

1. Soil Bearing Capacity: Presumptive soil bearing pressures of 2,000, 3,000, and 4,000 psf were used, representing common ranges encountered in typical Midwestern soil conditions, refer to *IBC 2021 Table 1806.2* (International Code Council, 2021). These values were selected to demonstrate the sensitivity of foundation dimensions to soil stiffness and bearing strength.
2. Concrete Compressive Strength (f'_c): Two strength levels, 3,000 psi and 4,000 psi, were applied for each soil bearing scenario. These values represent the most frequently specified concrete strengths for shallow foundations per *ACI 318-19(22) Table 19.3.2.1*

and correspond to exposure class F0 (non-freezing conditions) (American Concrete Institute, 2022).

3. Axial Column Loading (P_u): Four ultimate load cases (50, 100, 200, and 300 kips) were applied to represent a range of structural scenarios, from low-rise building columns to mid-range industrial supports.

When combined, these three parameters produced 48 total design cases (24 plain and 24 reinforced), ensuring that each result could be compared under identical site and loading conditions. The following discussion addresses trends in footing geometry, flexural and shear performance, and material efficiency.

4.2 Concrete Volume and Depth Relationships

The following two sections within this chapter have figures associated with the volumes of all 48 concrete footings designed for this report. Figure 4-2 - Plain Concrete Footing Volumes vs. Applied Ultimate Load and Figure 4-3 - Reinforced Concrete Footing Volumes vs. Applied Ultimate Load show volumes exclusive to plain and reinforced designs respectively. When reading the tables, the colored bars are associated with the site soil conditions and concrete compressive strengths that can be compared across plain and reinforced designs. Those bars are titled “Volume x000-x.0” where the first value is the soil bearing capacity in pound per square foot (psf), and the second value is the concrete compressive strength in kip per square inch (ksi). Those are organized on the right side of each figure.

4.2.1 Plain Concrete Footings

Plain concrete footings exhibited the largest variability in section thickness, primarily due to the direct correlation between bending resistance and section modulus. For a uniform loading condition, footing area is proportional to the applied load divided by the allowable soil bearing

pressure. Because plain footings resist tension solely through concrete's limited tensile strength, the design depth (h) becomes the critical factor for achieving adequate flexural capacity.

Across all combinations of soil and loading, plain footing depths ranged between 10 and 35 inches. At the lowest soil bearing (2,000 psf) and highest loading (300 kips), the section thickness nearly tripled compared to low-load conditions. The considerable volume increase is represented in Figure 4-2, where every footing condition is gathered together by their ultimate axial load applied. This visually shows the substantial disparity between lower loads against higher loads regardless of the difference caused by site conditions, though favorable site conditions do significantly mitigate the final volumes. This significant increase in concrete volume translates directly into higher costs and greater embodied carbon emissions from cement production.

Despite this, plain footings stand out as structurally viable and environmentally beneficial for low to moderate loads. Additionally, they can be advantageous in locations where reinforcement is difficult to source or corrosion protection is a concern.

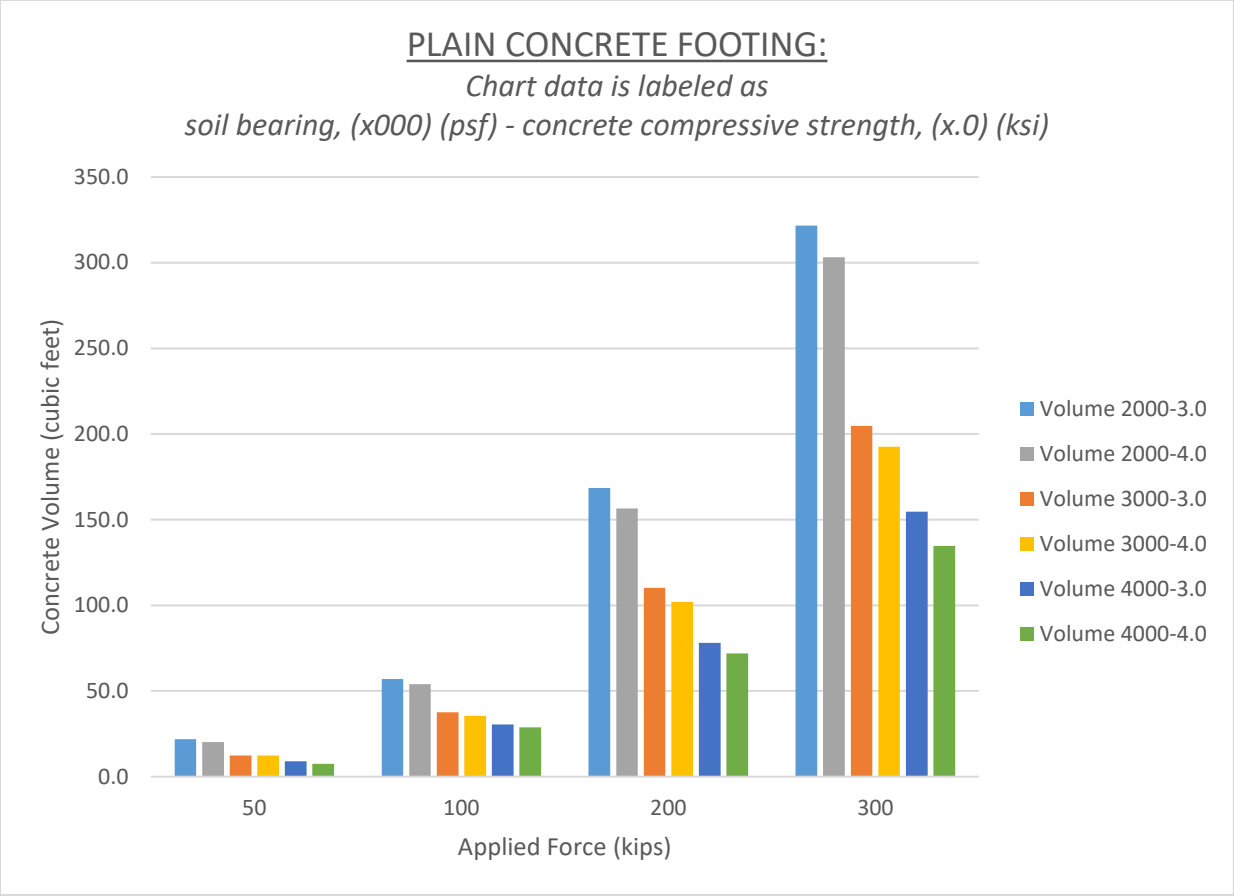


Figure 4-2 - Plain Concrete Footing Volumes vs. Applied Ultimate Load

4.2.2 Reinforced Concrete Footings

Reinforced concrete footings demonstrated higher efficiency due to the inclusion of tensile steel, which reduces the required section thickness for the same applied loads. The depth range for reinforced sections was 10 to 20 inches, typically half that of their plain counterparts under intense loading conditions. For example, at 300-kip loading on 2,000 psf soil, the reinforced footing required a total thickness of 19 inches, compared to 35 inches for the plain concrete design. See the blue and grey bars within Figures 4-2 and 4-3. This reduction in depth not only minimizes material use but also lowers excavation requirements and accelerates construction schedules.

Conversely, lighter loading conditions have sections of similar or identical section depths to their plain section counterparts, and with the added caveat of an increase in width required for adequate development length in reinforced footings. This does not occur for all reinforced sections but does have an impact on many of the lightly loaded sections and especially sections in the 4,000 psf soil bearing capacity range. Those exact ranges were as follows:

- 3,000 psf soil bearing, 3,000 psi *f'c* and under 50 kips of loading;
- 3,000 psf soil bearing, 4,000 psi *f'c* and under 50 kips of loading;
- 4,000 psf soil bearing, 3,000 psi *f'c* and under 50 kips of loading; and
- 4,000 psf soil bearing, 4,000 psi *f'c* and under 50 kips and less of loading.

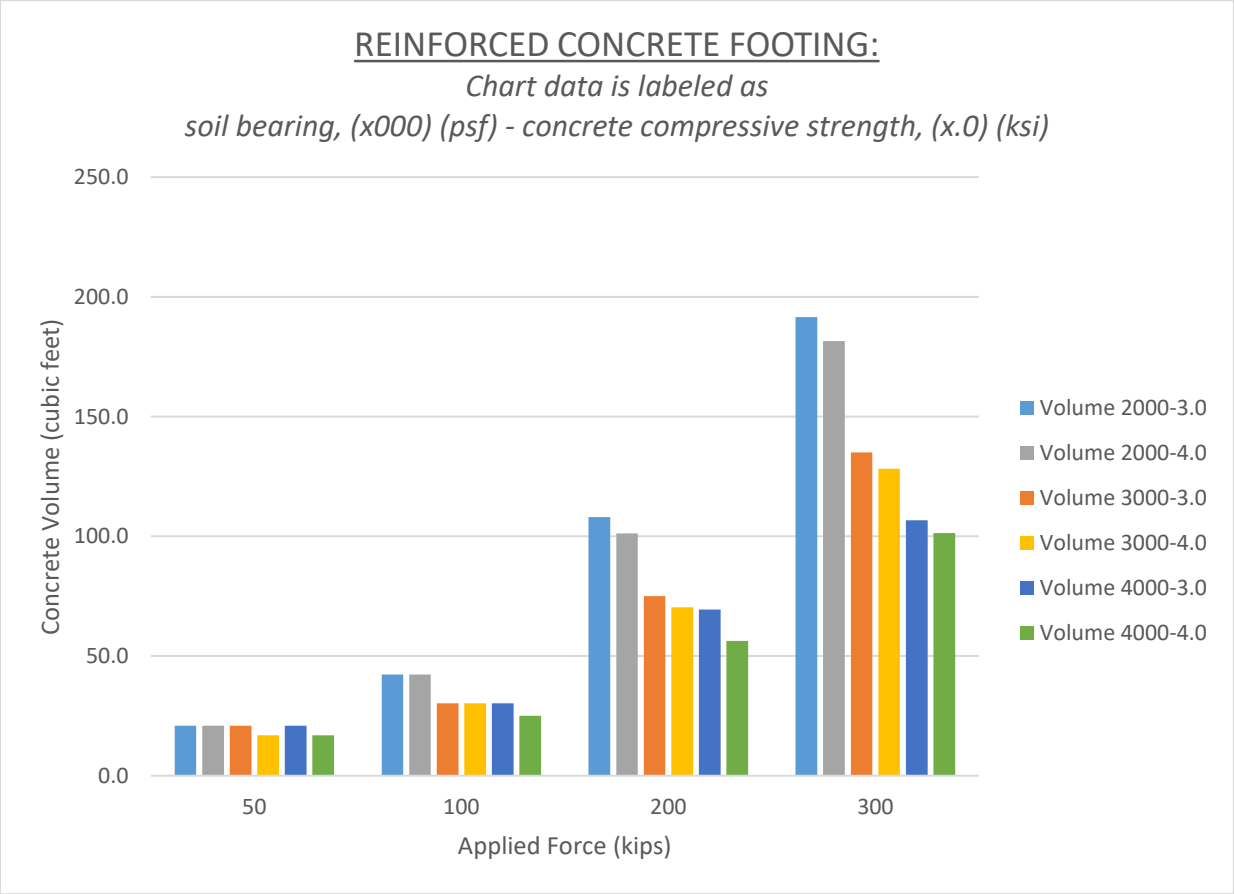


Figure 4-3 - Reinforced Concrete Footing Volumes vs. Applied Ultimate Load

The decreased depth and volume lead to notable reductions in embodied carbon, as concrete production is responsible for approximately 8% of global CO_2 emissions (Purton, 2024). Even when considering the added impact of reinforcing steel, the overall embodied carbon of reinforced designs was approximately on average 31% lower than equivalent plain sections due to the significant savings in concrete volume.

4.3 Reinforcing Steel Quantities and Efficiency

The inclusion of reinforcement introduces a balance between material cost, constructability, and sustainability. Per *ACI 318-19(22) Section 7.7.2.4*, bar spacing was governed by the lesser of 5*h* or 18 inches, and cover requirements followed *Table 20.5.1.3.1*, specifying 3 inches minimum when cast against soil.

The bar sizes evaluated ranged between No. 5 and No. 8, resulting in total steel areas (A_s) between 1.6 and 2.1 in², depending on design conditions. Heavier load cases required up to nine No. 8 bars, while lighter cases could be adequately reinforced with three No. 5 bars. Although the addition of steel marginally increases fabrication and installation costs, it provides significant savings in footing thickness, concrete volume, and construction time.

Across all reinforced examples, flexural utilization did not exceed 83%, even under extreme loading conditions. This conservative margin confirms compliance with *ACI 318-19(22) Section 22.3*, ensuring ductile failure modes and reserve capacity under overload or redistribution of stresses.

From an environmental perspective, the embodied carbon contribution of reinforcement when modeled using SE2050 or EC3 databases is minimal compared to that of concrete (Building Transparency, 2025); (Structural Engineering Institute, 2024). Thus, optimized

reinforcement layouts that minimize total concrete use are both structurally and environmentally favorable.

4.4 Shear Performance Analysis

4.4.1 One-Way Shear

In reinforced footings, one-way shear was the governing limit state for the majority of cases. Design checks were conducted per *ACI 318-19(22) Table 22.5.5.1(c)* (no shear reinforcement), using the steel ratio (ρ_w) and effective depth (d). With reinforcement located approximately two-thirds of the section depth from the compression face, as recommended in *R22.5.5.1*, optimized depths were determined iteratively to balance strength and material efficiency.

The highest one-way shear utilization occurred under 200 kips loading on 3,000 psf soil using 4,000 psi concrete, achieving 100% utilization at a total thickness of 15 inches. In contrast, the equivalent plain concrete section achieved only 22% utilization with a 25-inch thick section, demonstrating the drastic difference between bending capacity and shear resistance for plain sections. Across all reinforced section limit cases examined, one-way shear design is controlled in 24 of 24 cases, with an average utilization of 84%.

4.4.2 Two-Way (Punching) Shear

Two-way shear was evaluated following *ACI 318-19(22) Section 22.6* using *Table 22.6.5.2(a)* for reinforced concrete contributions without shear reinforcement. The governing condition (300 kips on 2,000 psf soil at 3,000 psi f'_c) resulted in a 93% utilization of punching shear capacity. Although this load case produced near-limit performance, the section remained within the allowable limits, verifying code compliance. In all other combinations, punching shear utilization ranged between 35% and 89%, typically higher than flexural utilization.

Flexural demand was greater in only three examples over two-way shear, but less critical than one-way shear in all designs.

These results indicate that punching shear effects are secondary but remain an essential check for local failure at column–footing interfaces. This same outlook applies to the plain sections where two-way shear was a secondary concern to bending. In plain sections, however, greater effect due to two-way shear was experienced in the footing over one-way shear.

4.5 Bearing Strength Evaluation

Bearing capacity was assessed according to *ACI 318-19(22) Section 22.8.3.2*, using both equations (a) and (b) from *Table 22.8.3.2*. Across all designs, equation (b), the case where the supporting surface is larger than the loaded area, governed. Nominal bearing strengths ranged from 848 to 1131 kips, with maximum utilizations below 35% even under the highest loads. These results confirm that bearing rarely governs footing design within the tested range, and that both plain and reinforced sections possess ample compressive capacity.

4.6 Plain Concrete Performance

For plain concrete footings, flexure governed all 24 design examples, consistent with *ACI 318-19(22) Section 14.5.2*. At 200-kip load, 4,000 psf soil, and 4,000 psi f'_c, the nominal moment utilization reached 99%, pushing the limits of the sections 24 in. to maximum material efficiency. Nineteen of the 24 cases exhibited utilizations exceeding 90%, attempting to be as efficient in resisting bending so that section depth and overall material quantity can be kept as low as possible.

Shear checks confirmed that plain concrete performs exceptionally well in shear, with one-way utilization below 27% and two-way utilization below 44% across all examples. The large effective depth of plain sections provides substantial shear capacity, although this comes at

the expense of material and cost efficiency. Bearing performance was similar to reinforced designs, with the highest utilization, 38% occurring in 3,000 psi concrete under 300-kip loading.

An additional limitation arises from *ACI 318-19(22) Section 14.5.1.7*, which requires designers to deduct 2 inches from total depth when the footing is cast directly against soil. This effectively increases required thicknesses and further widens the efficiency gap between plain and reinforced designs.

4.7 Comparative Summary and Discussion

Aspect	Plain Concrete	Reinforced Concrete
Controlling Limit State	Flexure	One-way shear
Typical Thickness Range	10–35 in	10–20 in
Volume Savings	—	approx. 29% less concrete
Maximum Utilization (Flexure)	99%	83%
Maximum Utilization (Shear)	44%	100%
Bearing Utilization	≤38%	≤35%
Embodied Carbon	High	approx. 29% lower overall
Constructability	Simple, no steel	More complicated, more compact
Best Use Case	Low loads, remote sites	Moderate–high loads, efficiency-driven designs

Table 4-2 Full Results Highlights and Overview

In summary, reinforced concrete footings outperform plain concrete footings in nearly all measurable metrics on average: strength-to-weight ratio, construction efficiency, and sustainability performance. Reinforced designs provide substantial advantages for higher levels of modern construction demands, particularly when carbon reduction and optimized material use are priorities. Highlighting the typical thickness range stated within Table 4-2, the higher end value for plain sections of 35 inches compared to the higher end value of a reinforced section at 20 inches indicates that material savings. Bending capacities between plain and reinforced sections is where the potential advantage of plain concrete is lost. When a project site has poor soil conditions, anticipates high concentrations of axial force from the structure, or some combination of the two should not consider plain concrete footings as a method of reducing the embodied carbon of the structure.

What Table 4-2 does a poor job is displaying where plain concrete could be considered advantageous in reducing environmental impacts of a structure. Lower loading demand examples are overlooked when considering the averages of poor to optimal conditions. More specifically, projects with good soil conditions that offer high bearing capacities make plain sections more appealing. Within the 48 calculations, the case of a 50 kip axially loaded foundation comprised of 4,000 psi compressive strength concrete and bearing onto soil with a bearing capacity of 4,000 psf demonstrated a case where there is a net positive in reducing embodied carbon by the use of a plain, unformed concrete footing over the reinforced, unformed concrete footing. Both sections had an identical depth of 10 inches that yielded acceptable designs. The plain footing had a bending demand over capacity utilization of 95%, compared to the reinforced sections utilization of 33%, representing a case where the use of steel reinforcing does not improve the designs structural efficiency. With the lack of form work and increased

length and width due to development length requirements, the reinforced sections took on extra concrete to provide adequate reinforcing steel clear cover and length minimums. If formed, the sections would still be different due to the development length requirements. With the inclusion of steel and formwork, the plain section still has a lower embodied carbon based on cradle-to-gate measurements, while simultaneously making the reinforced section constructability more complicated. Additional research regarding the environmental impact of labor and installation of forms and steel could further add to the point that plain sections being superior in an instance like this.

Furthermore, four additional calculations were produced based on this result with a lower ultimate load, set at 25 kips. A plain, unformed footing was designed with a 10-inch section depth, and another with an 8-inch section depth (the code minimum). These were compared to both a formed and unformed reinforced section, each with a 10-inch section depth. It should be reminded that a 10-inch section is the shallowest that could be reasonably designed that also adheres to code minimum standards and cannot be any shallower. Either plain section has a clear advantage over the unformed reinforced section. Following the same reasoning mentioned in the previous paragraph, the 10-inch-deep designs have differing volumes due to the development length requirements, indicating that the reinforced section includes unnecessary materials and construction needs for a satisfactory design when compared to the plain section. These values are tabulated within Appendix C.10 of this report. Plain footings make efficient use of the materials allowed, reduce the embodied carbon through this efficiency, being unformed, and by removing the need for reinforcing shipping and installation, reduce the complexity of construction.

Chapter 5: Recommendations and Conclusions

The environmental performance of structural systems has become an increasingly important metric in the engineering design process, particularly as the Architecture, Engineering, and Construction (AEC) industry accounts for approximately 40% of total global CO_2 emissions through construction and lifecycle use (United Nations Environment Programme, 2020). Concrete, as the most widely used construction material, is responsible for a large portion of these emissions, primarily from cement production and the energy-intensive processes of material extraction, manufacturing, and transportation (Miller, John, Pacca, & Horvath, 2018). This chapter discusses the embodied carbon implications derived from the parametric study results, comparing plain and reinforced concrete footing systems under varying design conditions.

Embodied carbon is defined as the total greenhouse gas emissions (GHG) associated with a material or system over its life cycle, specifically those arising from material extraction, processing, manufacturing, transport, and construction (RICS, 2017). In this study, the embodied carbon of each footing design was quantified from cradle-to-gate, corresponding to modules A1–A3 of the life-cycle assessment (LCA) framework per EN 15978:2011 (European Committee for Standardization, 2011). This scope was chosen to isolate the impact of material production and transport, excluding operational and end-of-life emissions.

5.1 Calculation Methodology

To ensure standardization, embodied carbon calculations followed methods consistent with the Structural Engineers 2050 (SE 2050) Challenge and Carbon Leadership Forum's EC3 database (2024) (Structural Engineering Institute, 2024); (Building Transparency, 2025). These databases provide industry-average and manufacturer-specific Environmental Product

Declarations (EPDs) for common construction materials, allowing for direct comparison between design options. The measurement of embodied carbon is in a Global warming potential unit measured by *unit CO_2e*, essentially these are the emissions associated with the creation of this material per unit measurement (Structural Engineering Institute, 2024). They are as follows:

Material	Typical Embodied Carbon (lb CO_2e per unit)	Reference
Normal-weight concrete (3,000 psi)	16.76 lb CO_2e/ft^3	(Structural Engineering Institute, 2024)
Normal-weight concrete (4,000 psi)	20.55 lb CO_2e/ft^3	(Structural Engineering Institute, 2024)
Reinforcing steel (ASTM A615, Grade 60)	0.85 lb CO_2e/lb	(Structural Engineering Institute, 2024)

Table 5-3 Embodied Carbon Values of Materials and Transport

Each footing's embodied carbon was computed as the sum of the carbon contributions from concrete and reinforcing steel, adjusted for the volume or mass of materials used. The per unit values for each material is represented in Table 5-3 Embodied Carbon Values of Materials and Transport. As examined, concrete has a significantly higher embodied carbon value compared to any other material, and therefore concrete was the dominant contributor across all designs, accounting for 85-92% of total embodied emissions.

5.2 Influence of Concrete Volume and Strength

The most significant driver of embodied carbon in both plain and reinforced designs was the volume of concrete required to achieve structural adequacy. As discussed in Chapter 4, plain

concrete footings required greater thickness (up to 35 inches under high loads and weak soil conditions) compared to reinforced counterparts, which rarely exceeded 20 inches. The final total comparison of concrete volumes across all loads and soil conditions, but separated by concrete compressive strength, the average volume reduction is 29% for reinforced footings.

Increasing the concrete strength from 3,000 psi to 4,000 psi had a smaller, though measurable, effect on embodied carbon. Higher-strength concrete has approximately 12% greater carbon intensity per cubic foot and the volume savings were not enough typically to offset the increase in material emissions (Structural Engineering Institute, 2024). For instance, reinforced footings designed at f'_c = 4,000 psi consumed approximately 4.99 cubic feet less concrete than their 3,000 psi equivalents but had a net 14% increase in embodied carbon. Attempting to mitigate embodied carbon through strength increase and volume savings does not have the intended benefit at the scale of these calculations. Substantially better soil conditions are required to achieve notable volume and embodied carbon savings, yet such conditions cannot be achieved on most project sites.

The interplay between compressive strength and design volume illustrates the importance of design optimization: adjusting the concrete compressive strength can yield substantial amounts volume savings, but depending on the exact number, may or may not produce an intended reduction in embodied carbon.

5.3 Role of Reinforcing Steel

While the inclusion of reinforcing steel introduces additional embodied carbon, its overall effect is comparatively minor. Steel's high carbon intensity is outweighed by the reduction in concrete volume and associated emissions. For example, in high-load cases where reinforced designs required roughly 200-250 lb of reinforcing steel, the corresponding increase in embodied

carbon was only 0.07–0.08 lb CO_2e, compared to 18.32–20.77 lb CO_2e/ft^3 saved from reduced concrete volume. Adding steel, compared to changing concrete strength, is able to save volume without the increased embodied carbon. In every comparison of plain sections with the same soil bearing capacities, 2,000, 3,000, and 4,000 psf, but differing compressive strengths, 3,000 and 4,000 psi, the section depth savings are one to two inches total across all tested cases of identical loading. Considering the two sets of embodied carbon evaluated, the slightly larger sections with lower compressive strengths had a lower embodied carbon compared to the sections with higher compressive strength at the same soil conditions and loading.

Comparing the lower compressive strength section to a reinforced section of identical conditions, the embodied carbon savings were significant under intense loading conditions and all ranges of soil bearing. The only time this is not true is in the previously described instances of low loading conditions paired with favorable soil bearing capacities. The additional consideration of increased section widths and lengths due to steel development length requirements, plain footings have greater embodied carbon savings against the inclusion of reinforcing steel attempting to reduce depth. Implementing these structurally efficient design practices will lead to more significant impacts in embodied carbon reduction.

Recycled steel, now standard in North American reinforcing bar production, can lower embodied carbon by up to 75% compared to virgin steel (World Steel Association, 2022). Efficient use of steel reinforcement, particularly when sourced from electric arc furnace (EAF) production routes, can result in both structural and environmental gains.

5.4 Construction Methods and Formwork Considerations

Formwork use also contributes to embodied carbon, albeit marginally relative to the total. As described in Chapter 2, foundation footings may be either earth-formed or formwork-cast,

depending on site conditions and code allowances per *IBC 2021 Section 1808.8.* This is permitted for both plain and reinforced sections. Formwork-cast designs typically require temporary timber or plywood molds, which add emissions from material fabrication and transport. However, because spread footing formwork is minimal and often reused, its embodied carbon rarely exceeds 3% of total emissions.

The earth-formed option potentially provides slight environmental advantages by eliminating the need for additional form materials altogether through the explored low-end loading conditions at 50 kips and less, especially in those areas with high soil bearing capacity. Considering too the minimum footing width and length requirements for development length of reinforcing bar also aids the explored earth-formed plain sections as replacements to reinforced sections. Those specific instances mentioned previously in this study were the reinforced sections at:

- 3,000 psf soil bearing, 3,000 psi *f'c* and under 50 kips of loading;
- 3,000 psf soil bearing, 4,000 psi *f'c* and under 50 kips of loading;
- 4,000 psf soil bearing, 3,000 psi *f'c* and under 50 kips of loading; and
- 4,000 psf soil bearing, 4,000 psi *f'c* and under 50 kips and less of loading.

These reinforced examples have worse embodied carbon values compared to their plain concrete counterparts. At the 4,000 psf soil bearing and 3,000 psi *f'c* condition, 100 kips of ultimate axial loading has a nearly identical embodied carbon as the plain unformed footing, there is a 0.05 lb CO2e advantage to the unformed reinforced footing.

Earth formed footings may be forbidden by a local authority having jurisdiction or have specific requirements within the *ACI 318-19(22)* such as *Section 14.3.2.1* and *Section 20.5.1.3.1* for plain and reinforced sections respectively.

5.5 Regional Material Sourcing and Transportation

The geographical context of material acquisition strongly influences embodied carbon outcomes. As discussed previously in Chapter 2 – Regional Design Considerations, aggregates in the Midwestern U.S. are typically sourced from local riverbeds and quarries, minimizing transport distances. This regional proximity keeps concrete's A4 (transport) emissions comparatively low—often below 5% of total embodied carbon.

However, reinforcing steel and admixtures may travel longer distances, depending on supplier availability. Future reductions in embodied carbon could be achieved through regional procurement strategies, including:

- Prioritizing ready-mix suppliers with published EPDs;
- Specifying cement blends with supplementary cementitious materials (SCMs) such as fly ash or slag; and
- Encouraging on-site batching for large projects to eliminate hauling emissions.

5.6 Comparative Summary of Embodied Carbon Results

Design Type	Average Concrete Volume (yd^3)	Average Steel Weight (lb)	Total Embodied Carbon (lb CO_2e)	% Reduction vs. Plain
Plain Concrete (3,000 psi)	3.72	0	747	—
Plain Concrete (4,000 psi)	3.45	0	850	—
Reinforced (3,000 psi)	2.52	130	528	29%
Reinforced (4,000 psi)	2.41	161	602	29%

Table 5-4 Average Embodied Carbon Values – All Cases Calculated

The data within Table 5-4 indicates a clear advantage for reinforced footings, as a summarized result of the 48 calculations from this report, achieving an average 29% reduction in embodied carbon compared to equivalent plain designs. These savings arise primarily from volume efficiency rather than material substitution. This aligns with the conclusion that intelligent structural optimization, rather than simply material avoidance, is the most effective path to decarbonization in concrete foundation systems.

This narrative becomes more nuanced as the exact final embodied carbons of every calculation are analyzed. The following Table 5-5, Embodied Carbon Totals of Plain and Reinforced Sections, displays the full range of final embodied carbon results between the plain and reinforced sections calculated. A negative value indicates that a plain section has less

embodied carbon than its reinforced counterpart. Here the exact conditions for replacing reinforced sections with plain sections are displayed.

q (psf)	f'_c (psi)	P_u (kips)	Plain (lb CO_2e)	Reinforced (lb CO_2e)	Difference in Embodied Carbon (lb CO_2e)
2000	3000	50	13.59	12.92	0.67
		100	35.30	26.21	9.10
		200	104.42	67.01	37.41
		300	199.17	118.88	80.29
2000	4000	50	15.38	15.84	-0.46
		100	41.02	32.13	8.89
		200	118.91	77.01	41.90
		300	230.30	138.06	92.24
3000	3000	50	7.59	12.92	-5.33
		100	23.23	18.77	4.46
		200	68.29	46.54	21.75
		300	126.79	83.77	43.02
3000	4000	50	9.31	12.83	-3.53
		100	26.90	23.01	3.89
		200	77.54	53.48	24.06
		300	146.35	97.56	48.80
4000	3000	50	5.57	12.92	-7.35
		100	18.81	18.77	0.05
		200	48.31	43.06	5.25
		300	95.81	66.19	29.62
4000	4000	50	5.70	12.83	-7.14
		100	21.79	19.01	2.78
		200	54.69	42.85	11.84
		300	102.24	77.09	25.14

Table 5-5 Embodied Carbon Totals of Plain and Reinforced Sections

An example of the plain section calculations are provided within Appendix A - Plain Concrete Design Example. Appendix B - Reinforced Concrete Design Example provides an example of the calculation performed for the reinforced section. Finally, the full range of plain and reinforced section results and capacities are within Appendix C - Calculation Result Tables, mainly C.1 – C.8. The last two portions of Appendix C are the summaries of the parametric

study highlight discussion from Chapter 3 (C.9) and the tabulated results of the 25 kip loading condition (C.10).

5.7 Discussion: Structural Efficiency as a Sustainability Tool

The results highlight a critical insight: structural design efficiency directly translates into carbon efficiency. Reducing section depth, optimizing reinforcement ratios, and calibrating concrete strength for realistic load conditions all contribute more to carbon reduction than incremental material substitutions alone. In this context, engineering judgment becomes an environmental tool, where rational design decisions yield measurable reductions in global emissions.

Moreover, the use of structural efficient design practices and incorporating parametric modeling tools can help identify material savings earlier in the design process. Specifically, incorporating embodied carbon as an optimization criterion alongside cost and performance enables engineers to pursue multi-objective design solutions that balance sustainability and structural reliability.

As engineers, it is a cornerstone of the of the profession to execute designs with the health and safety of others as the primary focus, and to be efficient in all aspects of design. By taking a brief moment within the design process, engineers possess the capability of creating small yet compoundable change for each project that they are a part of. This can be achieved while maintaining structural stability and reducing hours spent producing, transporting, and constructing foundation systems.

5.8 Future Implications

The findings of this study underscore the importance of integrating carbon assessment into early structural design phases, particularly for foundation systems that often represent a

significant portion of a building's embodied emissions. Future research and practice could expand upon this work by:

- Incorporating alternative low-carbon binders (e.g., limestone calcined clay cement or geopolymer concretes) or alternate reinforcing (microfiber reinforcing).
- Evaluating life-cycle carbon performance (A1–C4) rather than cradle-to-gate emissions alone.
- Consider a more expansive study of lower loading conditions (below 50 kips) for use in miscellaneous support cases.
- Addressing this study with the primary focus of construction optimization. Focusing on principals such as craft life safety (human life of staff on site), scheduling improvements, labor time and cost factors, etc.
- Applying probabilistic methods to capture uncertainty in emission factors and material sourcing.
- Using prefabricated concrete footings instead of cast-in-place footings.

5.9 Summary

In summary, the embodied carbon assessment demonstrates that reinforced concrete isolated footings are not only structurally superior but also environmentally advantageous within more rigorous design scenarios. While plain concrete footings are feasible for lighter structures (axial service level loads of 25 kips or less), they require excessive material volume, leading to higher embodied emissions for higher loading conditions (axial loads greater than 100 kips, service level). Reinforced designs, through efficient use of both steel and concrete, achieve a balanced reduction in both mass and carbon intensity.

If design cases are for more lighter or nuanced loading conditions, and especially if their site properties are favorable for structural design, the use of plain sections is able to reduce the embodied carbon of a structure and make the most of the sections structural efficiency in material use as well.

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Appendix A - Plain Concrete Design Example

Appendix Figure A.1. Page 1: Base Criteria

For this example we will use a column and footing from the one story category to examine the design process required for plain design. — ACI 318-19 (UNO)

Givens:

Steel column and baseplate bearing concentrically onto a square concrete spread footing. Assuming no overturning moment.

Soil Bearing Capacity (q):	4000 psf			
Axial Load (Pu):	100 kips	P=	69.500 kips	
Assumed Footing Depth (h):	18 in	OK		14.3.2.1
Square Column Dimensions (N):	16 in			
Material Factor (λ):	1 Normalweight Concrete			
Compressive Strength (f'c):	3000 psi			

Square Footing Design*: — 14.3.2.2

Area of Footing $A = \frac{P}{q}$	A:	17.375	ft^2
Square Footing Dimensions	Base/Length (L=B):	4.168	ft
	L/B:	4.500	
Footing Stability $q_u = \frac{\sum P_u}{A}$	Factored Soil Pressure (qu):	4.940	ksf

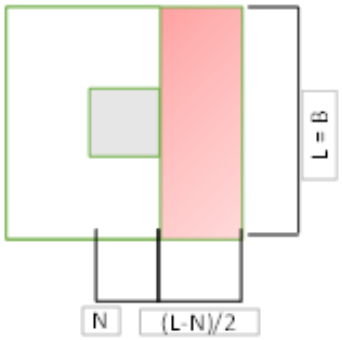
*values are rounded to nearest whole value

Appendix Figure A.2. Page 2: Bending Design Check

Flexure: 14.5.2

Location of flexure in the longitudinal direction 14.4.3.2

Table 14.4.3.2.1



Location of critical section for Mu is based on Table 14.4.3.2.1 and the design criteria established for this problem. A steel column with a base plate that has an intentionally unspecified column section and base plate geometery so that many combinations could equate to a 16" square contact area that is defined by the ACI 318-

N 1.333333 ft

Ultimate Moment (Mu): $M_u = \frac{q_u \left(\frac{L-N}{2}\right)^2 (B)}{2}$ Mu: 27.865 kip-ft

Flexure strength (Mn) shall be the lesser of Eq. (14.5.2.1a) calculated at the tension face and Eq. (14.5.2.1b) calculated at the compression face: 14.5.2.1

(a) $5\lambda\sqrt{f'_c}S_m$ (a) 631 kip-in (14.5.2.1a)

(b) $0.85f'_cS_m$ (b) 5875 kip-in (14.5.2.1b)

Sm = the elastic section modulus. Sm: 2304.0 in^3

ϕb: 0.6

ϕbMn: 31.549 kip-ft

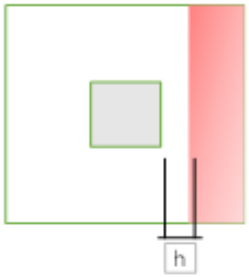
ϕbMn > Mu: OK

Percent Utilization % Utilized: 88%

Appendix Figure A.3. Page 3: One-way Shear Design Check

Factored One-Way Shear: 14.4.3.3

Location of one-way shear 14.4.3.3.1



(a) Table 14.4.3.2.1 Table 14.4.3.2.1
(b) Face of concretated loads

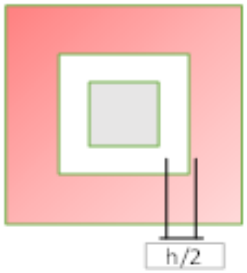
Following the procedure outlined in finding the critical location for moment and section 14.4.3.3.1, the critical shear area is located depth 'h' away from the previously defined location for steel columns/base

Ultimate Shear (Vu):	$V_u = \left(\frac{L}{2} - \frac{N}{2} - h\right) B\, q_u$	Vu:	1.853	k	14.5.5.1
Nominal shear strength (one-way):	$V_n = \frac{4}{3}\lambda\sqrt{f'_c}\, b_w h$	ϕv:	0.6		
		ϕvVn:	37.859	k	Table 14.5.5.1
		ϕvVn > Vu:	OK		14.5.1.1
statical moment:	Q = B*((h-2)/2)*((h-2)/2/2)	Q	1728	in^3	
max shear stress:	v = VQ/Ib	v	0.003	ksi	
		section area	864	in^2	
		max shear	2.8	kip	
		V,max ≥ Vu	OK		
Percent Utilization		% Utilized:	5%		

Appendix Figure A.4. Page 4: Two-way Shear Design Check

Factored Two-Way Shear: 14.4.3.4

Location of two-way shear 14.4.3.4.1



(a) Table 14.4.3.2.1
(b) Face of concretated loads
(c) Changes in footing thickness

Since we have a steel column framing into our spread footing, we will place the critical two-way shear area at a location 'h/2' from the face of our column, which satisfies both (a) and (b). We have no change in footing thickness.

Since we have a square column with a concentrated load, we can assume straight sides. 14.4.3.4.2

Perimeter (bo)	$b_o = 4(h+N)$	bo:	128	in	
Ultimate Shear (Vu)	$V_u = q_u\left((BL) - (h+N)^2\right)$	Vu:	60.378	k	
Nominal shear strength (two-way), Lesser of the following:	$V_n = \left(1+\frac{2}{\beta}\right)\left(\frac{4}{3}\lambda\sqrt{f'_c}b_o h\right)$	Vn:	448.694	k	Table 14.5.5.1
		β	1		
	$V_n = 2\left(\frac{4}{3}\lambda\sqrt{f'_c}b_o h\right)$	Vn:	299.130	k	
	β = 1 for square columns	ϕv:	0.6		
		ϕvVn:	179.478	k	
		ϕvVn > Vu:	OK		
Percent Utilization		% Utilized:	34%		

Appendix Figure A.5. Page 5: Bearing Design Check

Bearing:

Bearing (Bn) capacity in accordance with Table 14.5.6.1 — 14.5.6.1

Ultimate bearing based on maximum load faced by the column, in this case it is provided as Pu.

Supporting surface is wider on all sides than the loaded area. Lesser of: — Table 14.5.6.1

$$A_2 = min\left(L, N + \left(2(2h)\right)\right)^2$$

A_2: 20.3 ft^2

$A_{2,Max}$: 20.3 ft^2

(a) $\sqrt{\frac{A_2}{A_1}}\left(0.85f'_cA_1\right)$ (a) 2203.2 k

(b) $2\left(0.85f'_cA_1\right)$ (b) 1306 k

A1 = the area of the column
A2 = the area of the footing supporting the column and distributing the load

ϕ: 0.6

ϕBn: 783.360 k

ϕBn > Bu: OK

Percent Utilization — % Utilized: 13%

Appendix B - Reinforced Concrete Design Example

Appendix Figure B.1. Page 1: Base Criteria

For this example we will use a column and footing from the one story category to examine the design process required for reinforced design. — ACI 318-19 (UNO)

Givens:

Steel plate bearing concentrically onto a square concrete spread footing. Assuming no overturning moment.

Soil Bearing Capacity (q):	4000	psf			
Axial Load (Pu):	100	kips	P,ASD:	69.500	kips
Assumed Footing Depth (h):	12	in			
Reinf. Clearance from Bottom:	3.5	in			
Side Cover:	3	in	Formed: No		Table 20.5.1.3.1
Reinforcement Depth (d):	8.125	in	*OK*		13.3.1.2
Square Column Dimensions (N):	16	in			
Material Factor (λ):	1	Normalweight Concrete			
Compressive Strength (f'c):	3000	psi			
Reinforcement Yield Strength (fy):	60000	psi			

Square Footing Design: — 14.3.2.2

Area of Footing $A = \frac{P}{q}$ — A: 17.375 ft^2

Square Footing Dimensions — Base/Length (L=B): 4.668 ft

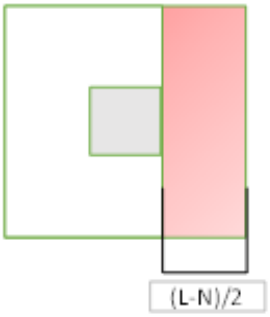
Rounded L/B & Max L/B or B,reqd: 5.000 5.500

Footing Stability $q_u = \frac{\sum P_u}{A}$ — Factored Soil Pressure (qu): 4.000 ksf

Appendix Figure B.2. Page 2: Bending Design Check

Flexure: 22.3

Location of flexure in the longitudinal direction 22.3.1.1



Location of critical section for Mu based on Table 13.2.7.1 and the design criteria established for this problem. A steel column and base plate that is purposly ambigious as to allow numerous design options for a 16" square critical area per the ACI 318-19 code. Table 13.2.7.1

N 1.333333 ft

Ultimate Moment (Mu): $M_u = \frac{q_u \left(\frac{L-N}{2}\right)^2 (B)}{2}$ Mu: 33.611 kip-ft 13.2.7.1

Moment Couple: $C = 0.85 f'_c b a$ 22.2.1.1

$T = A_s f_y$ 22.2.2.4

22.2.2.4.1

$R_n = \frac{M_u}{\phi b d^2}$ Rn: 113.1419 psi

$\rho = \frac{0.85 f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right)$ rho: 0.0019

rho,min: 0.0033

$A_s = \rho b d$ As: 1.625 in^2

Spacing and Number of Bars: Max. Spacing: Lesser 5h or 18": 18 in 7.7.2.4

Chosen: 18 in

# of Bars: 4.000

use: 4.000

Req'd Area of One Bar at Selected Spacing: 0.406 in^2

Bar #	Area of One Bar	# of Bars	As,prov. (in^2)	As,prov > As,req'd
5	0.310	4	1.24	N.G.
6	0.440	3	1.32	N.G.
6	0.440	4	1.76	OK

Appendix B

Use 4#6 As: 1.760 in^2

Minimum Area of Steel: $A_{s,min} = 0.0018 A_g$ As,min: 1.296 in^2 7.6.1.1

Appendix Figure B.3. Page 3: Bending Design Check cont.

Flexure (cont.):					22.3
Minimum Required Strength:	$\phi_b M_n \geq M_u$	Mu/ϕb:	37.346	kip-ft	7.5.1.1(a) 7.5.1.2
Determine adequate steel strain:	$a = \frac{A_s f_y}{0.85 f'_c b}$	a:	0.690	in	
	$c = \frac{a}{\beta_1}$	c:	0.767	in	(22.2.2.4.1) *rearranged*
		$\beta 1$:	0.9		Table 22.2.2.4.3
	$\varepsilon_t = \frac{\varepsilon_c}{c}(d - c)$	εt:	0.0288	>0.005	R21.2.2
		εty:	0.002		21.2.2.1
		εc:	0.003	max.	22.2.2.1
Nominal Strength:	$\phi_b M_n = A_s f_y \left(d - \frac{a}{2}\right)$	ϕb:	0.9		21.2
		ϕbMn:	61.617	kip-ft	22.2.1.1
	$\phi_b M_n \geq M_u$	ϕbMn ≥ Mu:	OK		
Percent Utilization		% Utilized:	55%		
Weight of Bars (One Direction):					
Length of Reinforcing		L,reinf.	5.000	ft	
Number of Bars		# of Bars	5.000		
Total Length		L,total reinf.	25.000	ft	
Bar Selected		Bar #	6		
Total Weight		Wt of Bar,total	37.550	lb	
			0.01878	ton	

Appendix Figure B.4. Page 4: Development Length Design and Pad Size Check

Development Length:

Development Length Equation

$$l_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \psi_g}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b$$

	ψt	1.000	Bottom Reinf.	(25.4.2.4b)
	ψe	1.000	Uncoated	
	ψs	0.800	#6 Bar	
	ψg	1.000	Grade 60	
	cb	3.000	in	
	Ktr	0	Simplify	
	db	0.750	in	
	ld	19.718	in	
Round the ld value to the next highest whole number	ld,use	20.000	in	

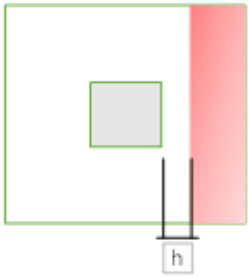
Required Width of Footing

$$B_{reqd} = CEILING(2 * l_{d,use} + N + 2 * c_b, 0.5)$$

Rounded to the next increment of 6 inches

B,$reqd$ 5.500 ft

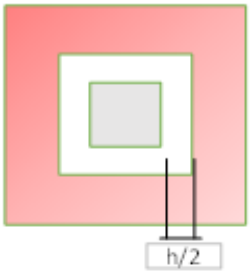
Appendix Figure B.5. Page 5: One-way Shear Design Check

Factored One-Way Shear:		22.5
Nominal one-way shear strength at a section, Vn, shall be calculated by:	$V_n = V_c + V_s$	(22.5.1.1)
Designing without shear reinforcement, Vs = 0.		
Vc for nonprestressed members (per Table 22.5.5.1):	$V_c = 8\lambda(\rho_w)^{1/3}\sqrt{f'_c}\,b_w d$	22.5.5.1(c)
	ρw = As/bd	
	$V_{c,max} = 5\lambda\sqrt{f'_c}\,b_w d$	22.5.5.1.1
	Vc,max: 133.507 k	
Ultimate Shear (Vu)	$V_u = \left(\frac{L}{2} - \frac{c}{2} - d\right) B q_u$	
	Vu: 23.125 k	
Set our nominal shear strength with our reduction factor applied equal to our ultimate shear, rearrange the equation and solve for our shear strength of concrete. Refine our depth assumptions from earlier if necessary to meet proper shear strength:	$\phi_v V_c = \phi_v 8\lambda(\rho_w)^{1/3}\sqrt{f'_c}\,b_w d$	
	ϕv: 0.75	Table 21.2.1(b)
	ϕvVc: 24.577 k	
	ϕvVn > Vu: OK	
Percent Utilization	% Utilized: 94%	
The assumed initial height of footing is valid, and may be refined for a more efficient design.		

Appendix Figure B.6. Page 6: Two-way Shear Design Check

Factored Two-Way Shear: *(see 22.6.5.1 to 22.3.5.2 to Table 22.6.5.2)* — 22.6

Location of two-way shear — 22.6.4.1



(a) Edges or corners of columns, concentrated loads, or reaction areas

(b) Changes in slab or footing thickness, such as edges of captials, drop panels, or shear caps.

We have a square column and concentrated loads, assume straight sides. — 22.6.4.1.1

Nominal shear strength for two-way members without shear reinforcement shall be calculated by: $v_n = v_c$ — (22.6.1.2)

Ultimate Shear (Vu):	$V_u = q_u\left((BL) - (d+c)^2\right)$	Vu:	83.833 k	
Perimeter (bo):	$b_o = 4(d+c)$	bo:	96.5 in	22.6.4
Nominal shear stress (two-way), Lesser of the following:	$v_c = 4\lambda_s\lambda\sqrt{f'c}$	vc:	219.089 psi	Table 22.6.5.2
	$v_c = \left(2 + \frac{4}{\beta}\right)\left(\lambda_s\lambda\sqrt{f'c}\right)$	vc:	328.634 psi	
		β	1	
	$v_c = \left(2 + \frac{\alpha_s d}{b_o}\right)\left(\lambda_s\lambda\sqrt{f'c}\right)$	vc:	294.011 psi	
		αs:	40	
	β = 1 for square columns			
	αs = 40 for interior columns	vc:	219.089 psi	

Take the shear stress times the perimeter (bo) and depth (d):

Vc:	171.779	k
ϕv:	0.75	
ϕvVc:	128.835	k
ϕvVn > Vu:	OK	

Percent Utilization — % Utilized: 65%

Appendix Figure B.7. Page 7: Bearing Design Check

Bearing: 22.8

Bearing (Bn) capacity in accordance with Table 22.8.3.2 — 22.8.3.2

Ultimate bearing based on maximum load faced by the column, in this case it is provided as Pu.

Supporting surface is wider on all sides than the loaded area. Lesser of: — Table 22.8.3.2

$$A_2 = min\left(L, N + \left(2(2h)\right)\right)^2$$

A_2: 25.0 ft^2

$A_{2,Max}$: 25.0 ft^2

(a) $\sqrt{\frac{A_2}{A_1}}\left(0.85f'_cA_1\right)$ (a) 2448 k

(b) $2\left(0.85f'_cA_1\right)$ (b) 1306 k

A1 = the area of the column
A2 = the area of the lower base of the largest frustum of a pyramid, completely within in the support and having its upper base equal to the loaded area.

ϕ: 0.65

ϕBn: 848.640 k

ϕBn > Bu: OK

Percent Utilization — % Utilized: 12%

Appendix C - Calculation Result Tables

Appendix Figure C.1. Plain Concrete – Material Results

Plain Concrete Material Results							
q (psf)	f'_c (psi)	P_u (kips)	P (kips)	Base/ Length (ft)	Height (in)	Volume (ft^3)	lb CO2e Concrete
2000	3000	50	34.8	4.5	13.0	21.9	13.59
		100	69.5	6.0	19.0	57.0	35.30
		200	138.9	8.5	28.0	168.6	104.42
		300	208.4	10.5	35.0	321.6	199.17
2000	4000	50	34.8	4.5	12.0	20.3	15.38
		100	69.5	6.0	18.0	54.0	41.02
		200	138.9	8.5	26.0	156.5	118.91
		300	208.4	10.5	33.0	303.2	230.30
3000	3000	50	34.8	3.5	12.0	12.3	7.59
		100	69.5	5.0	18.0	37.5	23.23
		200	138.9	7.0	27.0	110.3	68.29
		300	208.4	8.5	34.0	204.7	126.79
3000	4000	50	34.8	3.5	12.0	12.3	9.31
		100	69.5	5.0	17.0	35.4	26.90
		200	138.9	7.0	25.0	102.1	77.54
		300	208.4	8.5	32.0	192.7	146.35
4000	3000	50	34.8	3.0	12.0	9.0	5.57
		100	69.5	4.5	18.0	30.4	18.81
		200	138.9	6.0	26.0	78.0	48.31
		300	208.4	7.5	33.0	154.7	95.81
4000	4000	50	34.8	3.0	10.0	7.5	5.70
		100	69.5	4.5	17.0	28.7	21.79
		200	138.9	6.0	24.0	72.0	54.69
		300	208.4	7.5	31.0	134.6	102.24

Appendix Figure C.2. Plain Concrete – Moment Capacity Results and Utilization

Plain Concrete Strength Results (Moment)					
q (psf)	f'_c (psi)	P_u (kips)	M_u (kip-ft)	$\phi_b M_n$ (kip-ft)	Utilization (%)
2000	3000	50	13.9	14.9	93%
		100	45.4	47.5	96%
		200	151.2	157.4	96%
		300	301.1	313.1	96%
2000	4000	50	13.9	14.2	98%
		100	45.4	48.6	93%
		200	151.2	154.8	98%
		300	301.1	319.1	94%
3000	3000	50	8.4	9.6	88%
		100	33.6	35.1	96%
		200	114.9	119.8	96%
		300	227.0	238.4	95%
3000	4000	50	8.4	11.1	76%
		100	33.6	35.6	94%
		200	114.9	117.1	98%
		300	227.0	241.9	94%
4000	3000	50	5.8	8.2	70%
		100	27.9	31.5	88%
		200	90.8	94.6	96%
		300	190.4	197.4	96%
4000	4000	50	5.8	6.1	95%
		100	27.9	32.0	87%
		200	90.8	91.8	99%
		300	190.4	199.5	95%

Appendix Figure C.3. Plain Concrete – One-way Shear Capacity Results and Utilization

Plain Concrete Strength Results (One-way Shear)					
q (psf)	f'_c (psi)	P_u (kips)	V_u (kip)	$\phi_v V_c$ (kip)	Utilization (%)
2000	3000	50	5.6	26.0	21%
		100	12.5	53.6	23%
		200	29.4	116.2	25%
		300	47.8	182.2	26%
2000	4000	50	6.5	27.3	24%
		100	13.9	58.3	24%
		200	33.4	123.9	27%
		300	52.6	197.6	27%
3000	3000	50	1.2	18.4	6%
		100	6.7	42.1	16%
		200	16.7	92.0	18%
		300	26.5	143.0	19%
3000	4000	50	1.2	18.4	6%
		100	8.3	42.1	18%
		200	21.5	92.0	22%
		300	32.4	143.0	21%
4000	3000	50	-2.8	15.8	-18%
		100	1.9	37.9	5%
		200	5.6	75.7	7%
		300	13.4	122.3	11%
4000	4000	50	0.0	14.6	0%
		100	3.7	41.0	9%
		200	11.1	80.1	14%
		300	20.0	132.1	15%

Appendix Figure C.4. Plain Concrete – Two-way Shear Capacity Results and Utilization

Plain Concrete Strength Results (Two-way Shear)					
q (psf)	f'_c (psi)	P_u (kips)	V_u (kip)	$\phi_v V_c$ (kip)	Utilization (%)
2000	3000	50	35.6	104.1	34%
		100	76.4	196.7	39%
		200	162.9	382.8	43%
		300	251.7	566.8	44%
2000	4000	50	36.6	105.2	35%
		100	77.8	207.2	38%
		200	166.2	388.6	43%
		300	255.5	589.8	43%
3000	3000	50	27.8	91.1	31%
		100	67.9	179.5	38%
		200	147.9	359.3	41%
		300	228.3	538.4	42%
3000	4000	50	27.8	105.2	26%
		100	69.8	188.2	37%
		200	152.7	363.1	42%
		300	234.0	558.6	42%
4000	3000	50	19.8	91.1	31%
		100	60.4	179.5	38%
		200	132.1	336.5	41%
		300	211.3	510.7	42%
4000	4000	50	23.9	77.7	31%
		100	62.7	188.2	33%
		200	138.4	338.4	41%
		300	218.5	528.2	41%

Appendix Figure C.5. Reinforced Concrete – Material Results

Reinforced Concrete Material Results							
q (psf)	f'_c (psi)	P_u (kips)	P (kips)	Base/ Length (ft)	Height (in)	Volume (ft^3)	lb CO2e Concrete
2000	3000	50	34.8	5.0	10.0	20.83	12.90
		100	69.5	6.5	12.0	42.25	26.17
		200	138.9	9.0	16.0	108.00	66.89
		300	208.4	11.0	19.0	191.58	118.66
2000	4000	50	34.8	5.0	10.0	20.83	15.83
		100	69.5	6.5	12.0	42.25	32.09
		200	138.9	9.0	15.0	101.25	76.91
		300	208.4	11.0	18.0	181.50	137.87
3000	3000	50	34.8	5.0	10.0	20.83	12.90
		100	69.5	5.5	12.0	30.25	18.74
		200	138.9	7.5	16.0	75.00	46.45
		300	208.4	9.0	20.0	135.00	83.62
3000	4000	50	34.8	4.5	10.0	16.88	12.82
		100	69.5	5.5	12.0	30.25	22.98
		200	138.9	7.5	15.0	70.31	53.41
		300	208.4	9.0	19.0	128.25	97.42
4000	3000	50	34.8	5.0	10.0	20.83	12.90
		100	69.5	5.5	12.0	30.25	18.74
		200	138.9	7.0	17.0	69.42	43.00
		300	208.4	8.0	20.0	106.67	66.07
4000	4000	50	34.8	4.5	10.0	16.88	12.82
		100	69.5	5.0	12.0	25.00	18.99
		200	138.9	6.5	16.0	56.33	42.79
		300	208.4	8.0	19.0	101.33	76.97

Appendix Figure C.6. Reinforced Concrete – Moment Capacity Results and Utilization

	Reinforced Concrete Strength Results (Moment)					
q (psf)	f'_c (psi)	P_u (kips)	M_u (kip-ft)	Bar Count & Size	$\phi_b M_n$ (kip-ft)	Utilization (%)
2000	3000	50	16.8	4 #5	33.2	51%
		100	51.4	5 #6	77.2	67%
		200	163.3	8 #7	249.3	66%
		300	318.6	9 #8	459.7	69%
2000	4000	50	16.8	4 #5	33.5	50%
		100	51.4	5 #6	78.0	66%
		200	163.3	7 #7	202.6	81%
		300	318.6	8 #8	386.1	83%
3000	3000	50	11.1	4 #5	32.8	34%
		100	39.5	5 #6	76.6	52%
		200	126.9	7 #7	217.6	58%
		300	245.3	8 #8	435.5	56%
3000	4000	50	11.1	4 #5	33.3	33%
		100	39.5	5 #6	77.5	51%
		200	126.9	6 #7	173.5	73%
		300	245.3	7 #8	362.0	68%
4000	3000	50	8.4	3 #5	24.8	34%
		100	33.6	4 #6	61.6	55%
		200	102.8	6 #7	202.8	51%
		300	208.4	7 #8	381.3	55%
4000	4000	50	8.4	3 #5	25.1	33%
		100	33.6	4 #6	62.3	54%
		200	102.8	6 #7	188.8	54%
		300	208.4	7 #8	360.6	58%

Appendix Figure C.7. Reinforced Concrete – One-way Shear Capacity Results and Utilization

Reinforced Concrete Strength Results (One-way Shear)					
q (psf)	f'_c (psi)	P_u (kips)	V_u (kip)	$\phi_v V_c$ (kip)	Utilization (%)
2000	3000	50	13.2	18.2	72%
		100	29.4	31.5	93%
		200	62.9	66.1	95%
		300	97.8	99.6	98%
2000	4000	50	13.2	21.1	63%
		100	29.4	36.4	81%
		200	64.7	68.9	94%
		300	100.0	105.7	95%
3000	3000	50	10.2	15.7	65%
		100	25.6	28.2	91%
		200	55.5	56.0	99%
		300	83.5	87.5	95%
3000	4000	50	10.2	18.1	56%
		100	25.6	32.6	79%
		200	57.7	58.0	100%
		300	86.3	92.6	93%
4000	3000	50	8.1	13.1	62%
		100	23.1	24.6	94%
		200	46.1	51.0	90%
		300	75.0	77.4	97%
4000	4000	50	8.1	15.1	54%
		100	23.1	28.4	81%
		200	48.6	55.8	87%
		300	78.2	85.6	91%

Appendix Figure C.8. Reinforced Concrete – Two-way Shear Capacity Results and Utilization

Reinforced Concrete Strength Results (Two-way Shear)					
q (psf)	f'_c (psi)	P_u (kips)	V_u (kip)	$\phi_v V_c$ (kip)	Utilization (%)
2000	3000	50	43.2	90.2	48%
		100	90.6	128.8	70%
		200	186.6	222.5	84%
		300	283.5	305.6	93%
2000	4000	50	43.2	104.2	41%
		100	90.6	148.8	61%
		200	187.5	227.2	83%
		300	284.6	318.8	89%
3000	3000	50	39.4	90.2	44%
		100	86.7	128.8	67%
		200	180.8	222.5	81%
		300	274.1	336.5	81%
3000	4000	50	39.4	104.2	38%
		100	86.7	148.8	58%
		200	182.1	227.2	80%
		300	275.8	352.9	78%
4000	3000	50	36.1	90.2	40%
		100	83.8	128.8	65%
		200	172.5	249.5	69%
		300	266.8	336.5	79%
4000	4000	50	36.1	104.2	35%
		100	83.8	148.8	56%
		200	174.3	256.9	68%
		300	268.9	352.9	76%

Appendix Figure C.9. Parametric Study Results - 100 kip Ultimate Loading

Parametric Study Results - 100 kip Ultimate Loading						
q (psf)	2000		3000		4000	
f'_c (psi)	3000		4000		3000	
	Plain	Reinf.	Plain	Reinf.	Plain	Reinf.
Base/ Length (ft)	6.0	6.5	5.0	5.5	4.5	5.5
Height (in)	19.0	12.0	17.0	12.0	18.0	12.0
Volume (ft^3)	57.0	42.3	35.4	30.3	30.4	30.3
lb CO_2e Concrete	35.30	26.17	26.9	22.98	18.8	18.74
lb CO_2e Steel	-	0.038	-	0.032	-	0.032
lb CO_2e Total	35.30	26.207	26.9	23.01	18.8	18.768
M_u (kip-ft)	45.4	51.4	33.6	39.5	27.9	33.6
Bar Count & Size	-	5 #6	-	5 #6	-	4 #6
$\phi_b M_n$ (kip-ft)	47.5	77.2	35.6	77.5	31.5	61.6
Utilization (%)	96%	67%	94%	51%	88%	55%
V_u – One-way (kip)	12.5	29.4	8.3	25.6	1.9	23.1
$\phi_v V_c$ – One-way (kip)	53.6	31.5	42.1	32.6	37.9	24.6
Utilization (%)	23%	93%	18%	79%	5%	94%
V_u – Two-way (kip)	76.4	90.6	69.8	86.7	60.4	83.8
$\phi_v V_c$ – Two-way (kip)	196.7	128.8	188.2	148.8	179.5	128.8
Utilization (%)	39%	70%	37%	58%	38%	65%

Appendix Figure C.10. Plain vs. Reinforced Concrete Section – 25 kip Ultimate Loading

Plain versus Reinforced Concrete Section (25 kip Loading)				
q (psf)	4000			
f'_c (psi)	4000			
P_u (kips)	25			
P (kips)	17.4			
	Plain Section		Reinforced Section	
Formwork	no	no	no	yes
Base/ Length (ft)	2.5	2.5	4.5	4.5
Height (in)	10.0	8.0	10.0	10.0
Volume (ft^3)	5.2	4.2	16.9	16.9
lb CO_2e Concrete	3.96	3.17	12.82	12.82
M_u (kip-ft)	1.7	1.7	2.9	1.7
Bar Count & Size	-	-	3 #5	3 #5
$\phi_b M_n$ (kip-ft)	5.1	2.8	24.9	24.7
Utilization (%)	34%	60%	12%	7%
V_u – One-way (kip)	-2.5	-0.8	2.6	0.7
$\phi_v V_c$ – One-way (kip)	12.1	9.1	13.6	12.1
Utilization (%)	-21%	-9%	19%	6%
V_u – Two-way (kip)	6.2	9.0	15.5	11.3
$\phi_v V_c$ – Two-way (kip)	77.7	53.4	104.2	104.2
Utilization (%)	8%	17%	15%	11%