BUILDING ENVELOPE INSPECTION AND REHABILITATION ENGINEERING



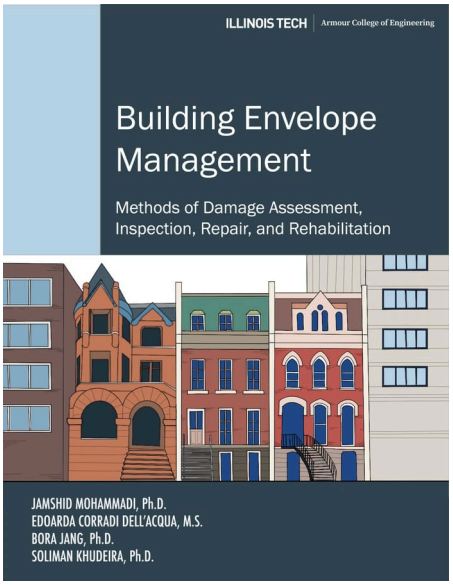
Jamshid Mohammadi, Edoarda Corradi Dell' Acqua, Bora Jang, and Soliman Khudeira

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BUILDING ENVELOPE MANAGEMENT: Methods of Damage assessment, Inspection, Repair and Rehabilitation

BUILDING ENVELOPE MANAGEMENT

Methods of Damage Assessment, Inspection, Repair and Rehabilitation



About the Book: This is a textbook intended to serve as a comprehensive reference for engineers, architects, and building professionals involved in the inspection, condition assessment, repair, and rehabilitation of building exteriors - collectively referred to as the building envelope.

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Preface

Overview

This book is the result of a collaborative effort between the Galvin Library and the Department of Civil, Architectural, and Environmental Engineering at the Illinois Institute of Technology (Illinois Tech), Chicago, Illinois, USA. The book is intended to serve as a comprehensive reference for engineers, architects, and building professionals involved in the inspection, condition assessment, repair, and rehabilitation of building exteriors - collectively referred to as the building envelope.

The term building envelope describes the physical barrier that separates the interior environment of a building from the external climate. While often colloquially associated with the façade, the building envelope encompasses a wider array of systems and components, including roofs, windows, doors, curtain walls, cladding assemblies, and in some cases, foundation elements. Throughout this book, the terms façade and building envelope may be used interchangeably for practical purposes; however, it is important to acknowledge that the building envelope refers to a more comprehensive set of assemblies beyond the visible exterior skin of a structure.

Building envelopes serve critical multi-functional roles in modern construction. Beyond defining the architectural identity and aesthetic character of a building, the envelope must also perform essential protective and environmental control functions. These include the regulation of heat, air, and moisture transfer, providing thermal comfort, sound insulation, weather resistance, and supporting energy efficiency and indoor air quality. Structurally, the envelope must exhibit durability, resilience, and resistance to environmental degradation, reducing the risk of failure that could endanger building occupants, passersby, or adjacent properties.

Given their importance to life safety and building performance, major urban jurisdictions have implemented legal requirements mandating periodic inspection and reporting of building envelopes, particularly for mid- and high-rise structures. These regulations are designed to ensure early identification of deterioration or structural failures that could pose a public hazard. Consequently, professionals working in the field must possess a deep understanding of the design intent, performance requirements, failure modes, and appropriate maintenance and repair practices associated with various envelope systems.

In this context, the term “building envelope engineering management” – as used in this book – refers to the full range of professional activities involved in ensuring the ongoing integrity, safety, and performance of the building envelope. This includes, but is not limited to:

- Routine and special inspections.
- Condition assessments and forensic evaluations.
- Development of repair and rehabilitation programs.
- Implementation of remedial measures in accordance with governing codes, standards, and jurisdictional regulations.

The contents of this book are organized into ten chapters and two appendices, each addressing a critical aspect of building envelope engineering and management.

Chapter 1 offers a general overview of building envelope systems, their functions, the roles and responsibilities of engineers and architects in envelope management, and a glossary of commonly used terminologies in the field.

Chapter 2 focuses on masonry veneer wall systems, including their performance characteristics, typical distress conditions, and failure mechanisms.

Chapter 3 covers concrete façades, reinforced masonry walls, and load-bearing wall systems, with emphasis on structural behavior and deterioration.

Chapter 4 addresses façade systems composed of terra cotta and stone blocks, stucco, and Exterior Insulation and Finish Systems (EIFS), discussing both historical and contemporary applications.

Chapter 5 discusses the administrative and procedural aspects of building envelope inspections, including project planning, report preparation, and compliance with municipal regulations.

Chapter 6 provides an overview of parapet walls and roof systems, including considerations related to green roofs, roof drainage,

and roofing membrane assemblies.

Chapter 7 examines ancillary structural components critical to envelope performance, such as shelf angles, lintels, girt beams, and support systems.

Chapter 8 introduces nondestructive evaluation (NDE) and monitoring technologies commonly employed in building envelope diagnostics, including thermography, ground-penetrating radar, and sensor-based monitoring.

Chapter 9 explores the integration of economic analysis and reserve studies into the decision-making process for envelope repair and maintenance planning.

Chapter 10 presents an emerging frontier in the field - exploring the use of Virtual Reality (VR) and Augmented Reality (AR) technologies to enhance envelope inspection, visualization, and documentation.

Appendix A includes detailed numerical illustrations and calculations relevant to building envelope assessments.

Appendix B provides a curated list of open-access resources, technical references, and supplemental reading materials to support further learning.

This book is intended to bridge the gap between theoretical understanding and practical application in the field of building envelope engineering. It emphasizes both technical rigor and real-world relevance, providing a framework for professionals to evaluate, manage, and extend the service life of exterior building systems safely and effectively. We hope it will serve as a valuable resource for practitioners, researchers, and students alike who are engaged in the evolving discipline of building envelope design, inspection, and rehabilitation and management.

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1: UNDERSTANDING BUILDING ENVELOPES, THEIR FUNCTIONS AND TYPES

Chapter Synopsis

This introductory chapter covers an overview of several key topics related to the inspection, repair and rehabilitation of building envelopes. It includes:

- Types of building envelopes.
- Architectural and engineering design issues pertaining to building envelopes.
- Types of deficiencies and distress conditions.
- Building envelope engineering management.
- Role and responsibilities of engineers and architects in inspection, repair and rehabilitation of building envelopes.
- Quality assurance and professional ethics.
- Capital development and investment returns on building envelope repair and maintenance.

Additionally, an illustrative nomenclature for common terms used in building envelope inspection, repair and rehabilitation is presented in this chapter.

1.1: Introductory Remarks

1.2: Types of Building Envelopes

1.3: Architectural and Engineering Design Issues Pertaining to Building Envelopes

1.4: Types of Deficiencies and Distress Conditions

1.5: Building Envelope Engineering Management

1.6: Role and Responsibilities of Engineers and Architects in Inspection, Repair and Rehabilitation of Building Envelopes

1.7: Quality Assurance and Professional Ethics

1.8: Capital Development and Investment Returns on Building Envelope Repair and Maintenance

1.9: Common Terminologies Used In Building Envelope Inspection

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1.1: Introductory Remarks

The term “building envelope” refers to the physical barrier between the interior and exterior environments of a structure. While it is commonly associated with the façade, the building envelope typically also encompasses other critical components such as the roof, windows, doors, and even in certain cases foundation elements. In many texts – including this book – the terms building envelope and façade are often used interchangeably. However, it is important to recognize that the building envelope encompasses a broader range of systems and assemblies beyond the visual façade.

As will be further elaborated in this chapter, building envelopes serve multiple essential functions. They contribute significantly to the aesthetic character of a building while also functioning as a protective barrier against external environmental conditions. These functions include regulating heat, air, and moisture flow, ensuring thermal comfort, acoustic insulation, weather resistance, and energy efficiency for the occupants. Additionally, the building envelope must provide structural integrity, resiliency, and resistance to deterioration, thereby minimizing risks of component failure that could pose hazards to occupants, pedestrians, and nearby structures.

Due to their critical role in public safety and building performance, many major municipalities have enacted regulations that mandate periodic inspection of building envelopes, particularly for buildings exceeding certain height thresholds. These regulatory frameworks are designed to ensure that deterioration or failure of envelope components is detected and mitigated before it leads to unsafe conditions. As such, a thorough understanding of the building envelope’s functions, potential modes of deterioration, and the appropriate maintenance and repair strategies is essential for effective building envelope engineering management.

The term "building envelope engineering management" (or simply building envelope management), as used in this book, refers to the comprehensive set of activities aimed at maintaining the integrity, performance, and safety of the building envelope. This includes – but is not limited to – routine inspections, condition assessments, diagnostic evaluations, and the prescription and implementation of remedial measures in accordance with applicable codes, standards, and jurisdictional mandates.

This chapter aims to provide an overview of the key functions of building envelopes, the expertise required for their effective management, and the common types of building envelope systems along with their respective damage mechanisms and vulnerabilities. A more detailed and systematic exploration of these topics – including inspection protocols, deterioration patterns, repair technologies, and regulatory considerations – are presented in the subsequent chapters.

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1.2: Types of Building Envelopes

Building envelopes comprise a diverse array of materials and architectural configurations, serving both functional and aesthetic roles. Broadly, they can be classified into veneer systems, curtain walls, load-bearing walls, and façades constructed from prefabricated panels. However, the specific design and performance characteristics of a building envelope are influenced by factors such as building type, height, intended use, and desired visual expression. As such, each envelope system must be tailored to meet the unique demands of its architectural and environmental context.

Building envelopes can be categorized into several distinct types, each characterized by unique structural and aesthetic properties. The following list highlights common envelope systems frequently employed in various building typologies:

- **Brick masonry**: This may be in the form of a veneer or the entire façade may also be a load-bearing system.
- **Prefabricated panels**
- **Tilt-up panels**: Popular in manufacturing and storage facilities.
- **Concrete masonry units (CMU)**: Mainly reinforced and used as a load-bearing system.
- **Stone masonry**
- **Façade made up of terra-cotta blocks**
- **Other materials**: Such as wood panels and weatherboarding, aluminum panels, etc.
- **All-concrete walls**: Almost always reinforced and used as a load-bearing system.
- **Adobe**: Made up of clay and similar materials.
- **Stucco or render**: A material made of plaster applied to façades made up of other materials.
- **Exterior insulation and finish system (EIFS)**
- **Green walls**
- **Glass panels**

Table 1.1 provides a detailed summary of the characteristics, construction methods, and relevant attributes of these envelope types. While each system exhibits distinct construction practices, vulnerability to distress, maintenance requirements, and repair strategies – as elaborated in subsequent chapters – they collectively fulfill the essential functions of enhancing building aesthetics, structural stability, durability, and environmental protection. Moreover, even within a single category, façades display considerable variation, reflecting unique design solutions tailored to specific architectural and functional demands.

Table 1.1 Characteristics of various types of building envelopes

Type	Composition	Characteristics	Other attributes
Brick masonry	Layer of bricks with mortar	Used as veneer or load-bearing wall, reinforced or unreinforced	Veneers require shelf angles for support
Prefabricated panels	Concrete	Used as curtain walls	Require proper structural support
Tilt-up panels	Reinforced concrete	Load-bearing system	Stand-alone system, applicable to low-rise buildings
Concrete masonry units (CMU)	Layers of hollow cement blocks	Mainly used as reinforced load-bearing wall	Low-rise applications, often form secondary walls behind façade

Stone masonry	Layers of stone blocks	Used as panels or load-bearing walls	If used as panels, they require proper structural anchors for installation
Terra-cotta	Layers of hollow blocks baked from clay and glazed on surface	Used for the entire façade or as ornamental pieces	Blocks require proper structural anchors for installation
Wood panels and weatherboarding	Usually in the form of planks overlapping each other	Protection against sunlight damage and bugs required	Usually low-rise applications
Aluminum and synthetics panels	Usually in the forms of planks	Often referred to as “sided” in industry	Usually in low-rise applications
All-concrete walls	Reinforced concrete	Load-bearing low height walls	May also be used in partial basements with portions exposed as façade and other portions under soil pressure acting as a retaining wall
Adobe	Made up of clay and like materials	Suitable for warm dry climates	Application usually in single family units
Stucco or render	Plaster applied to façades made up of CMU or brick blocks, metal, adobe, etc.	Plaster requires a supporting metal mesh or lath	Application usually in single family units
Exterior insulation and finish system (EIFS)	A layer of thin stucco-like material applied over insulation (all external)	Requires proper sealing around windows and other openings	Low-rise applications
Green walls	Vegetation grown on stand-alone trellises or directly on brick, concrete or CMU walls	Requires proper irrigation system and periodic trimming	Low-rise applications with consideration for aesthetics and energy conservation
Glass	Used as a curtain wall made up of glass panels	Requires special detailing to maintain stability and prevent damage	Popular in many modern low- and high-rises

Several kinds of building façades are shown in Figure 1.1 depicting the variety in types and appearances. Because of many different features that are generally unique to a specific building, each façade inspection project is different from another and requires attention to a variety of factors including the occupancy type, age, architectural design features, location, height, structural design, environmental conditions, history of repair and rehabilitation as well as the type and proximity of neighboring buildings.

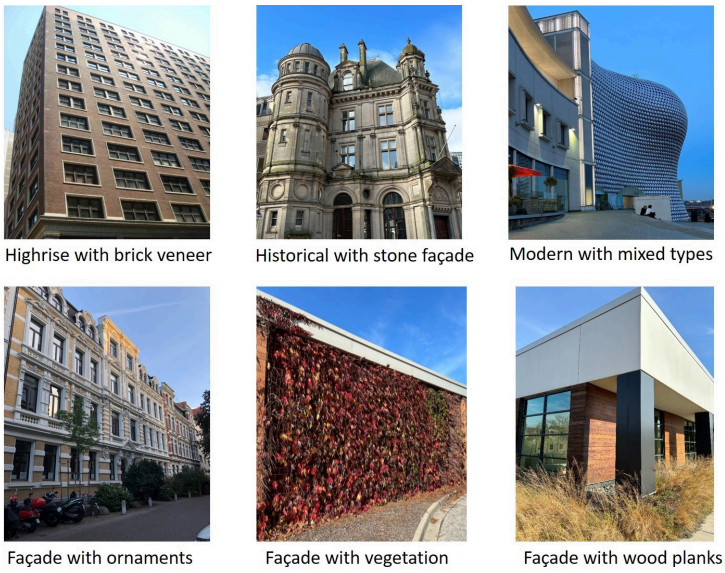


Figure 1.1 A sample of various kinds of façades (from personal collection of lead author)

(Six photos of façades: top – brick high-rise, old stone building, modern mixed façade; bottom – ornate façade, green wall, modern wood façade.)

Even within a class of building envelope types, variations are often expected, which will again be unique to the architecture of the building. For example, Figure 1.2 shows three buildings with glass façades. As seen in this figure, these buildings are very different; and as such, each one will require attention based on the kind of conditions the façades are exposed to, the architectural details, kinds of glass panel framing, and the blend of glass with other materials.



Figure 1.2 Glass building envelopes with different architectural details blended with other materials (from personal collection of lead author)

(Three photos of application of glass in façades blended with other kinds.)

Figure 1.3 presents a unique application of glazed tiles in creating artwork on a façade. This 31-story structure is renowned for its rainbow mural, which was designed by artist Millard Sheets and completed in 1968. The mural, measuring 286 feet (87 meters) high by 26 feet (8 meters) wide, was recognized as the world's tallest mosaic at the time of its construction. The rainbow mural consists of tiles that decorate the north and south façades of the tower, originally made up of over 16,000 hand-painted ceramic

tiles. In a restoration effort completed in 2014, the number of tiles was increased to over 31,000 and incorporating new LED lighting for visibility at night.



Figure 1.3 The Rainbow Tower with a façade made up of decorative tiles creating a mural (from personal collection of lead author)

(The Rainbow Tower in Hilton Hawaiian Village, Honolulu with painted tiles to form a rainbow image: a close-up image of tiles is shown to the right.)

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1.3: Architectural and Engineering Design Issues Pertaining to Building Envelopes

The envelope is a unique component of a building with a multifunctional nature that encompasses various areas of architecture and engineering, including aesthetic design, stability and resiliency, energy control, heat transfer, and protection against the surrounding environment. Looking more closely at these functions, it is evident that the involvement of various expertise is crucial in building envelope management, which includes inspection, condition assessment, repair and rehabilitation.

Figure 1.4 provides an overview of the multiple tasks involved in building envelope management, as well as the involvement of the three professions in the process.

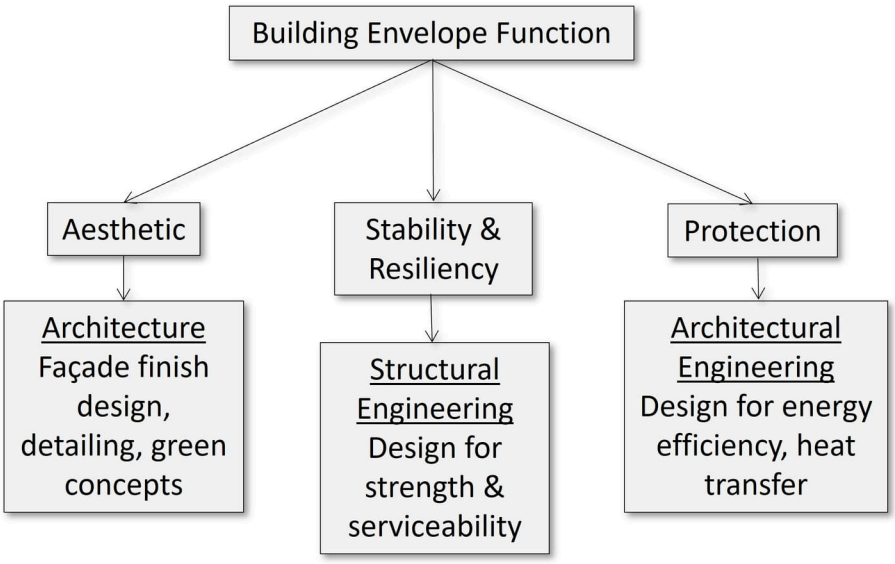


Figure 1.4 Functions of a building envelope and involvement of various professions in its inspection and rehabilitation

(Chart of building envelope functions: aesthetics (façade design, detailing, green concepts), stability (structural engineering), and protection (energy efficiency, heat transfer.)

Aesthetic design pertains to architecture. In most cases, the building envelope is expected to preserve its original design, which can be challenging when only a portion of the façade requires rehabilitation and reconstruction. Any repair and reconstruction must match the rest of the façade in terms of color, appearance, and aesthetics. Architectural considerations require a detailed investigation of repair and reconstruction strategies that are feasible and will not impair the aesthetic design. Decisions such as carefully removing certain elements of the façade and reusing them in the repair and reconstruction process are not uncommon.

Stability and resiliency pertain to structural engineering, which involves addressing issues like identifying the strength of various façade components and replacing load-bearing members with new ones that meet current loading regimes and governing design codes and standards. Since the work requires finding structural engineering solutions for existing structural conditions, identifying proper loading and component strength can be challenging. Consequently, reliance on load measurements over an extended period of time may be necessary. Additionally, determining the strength of materials in structural components may require sampling and nondestructive testing.

As shown in Figure 1.4, the building envelope protects against environmental conditions such as extreme heat, extreme cold, rainwater, and noise. These concerns fall within the scope of architectural engineering, where issues like energy control, heat transfer, and acoustics present significant design challenges in the repair, rehabilitation, and reconstruction of building envelopes.

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1.4: Types of Deficiencies and Distress Conditions

Deficiencies and distress in building envelopes can manifest in various forms of damage. Common examples include masonry cracks, mortar washout, inadequate anchoring of stone and terra-cotta blocks, bulging veneer walls, distorted glass panel frames, and corrosion in lintels and shelf angles. As expected, the occurrence of these issues largely depends on factors such as the type of building envelope, its components, material types, environmental conditions, building age, and the presence of unusual loadings.

Table 1.2 summarizes various common deficiency and distress conditions associated with different types of building envelopes, along with brief descriptions of their plausible causes. However, it is important to note that each case is unique, and assessing the structural condition of a building envelope requires a thorough investigation, which may involve non-destructive evaluation methods.

Table 1.3 Typical deficiencies and distress conditions in various kinds of building envelopes

Building Envelope Type	Some Common Deficiencies	Examples of Causes
Brick masonry	Crack in masonry; missing/damaged bricks; mortar washout; inadequate anchors behind wall; efflorescence	Weather condition; wear and tear; freeze/thaw cycles; rain; ice build-up
Prefabricated panels	Cracks and spalling in concrete; inadequate anchors	Weather conditions; wear and tear; construction/installation errors
Tilt-up panels	Cracks and spalling in concrete; corrosion of shear connection brackets (attaching adjacent panels); sealant and joints between panels worn/damaged	Weather conditions; age; construction/ installation errors; foundation settlement
Concrete masonry units (CMU)	Crack in masonry; missing/damaged CMU; mortar washout, inadequate anchors behind walls; efflorescence	Weather condition, wear and tear, freeze/thaw cycles; rain; ice build-up; lack of proper reinforcement
Stone masonry	Crack in masonry; missing/damaged stone; mortar washout; inadequate anchors for securing stone blocks	Weather condition; wear and tear; freeze/thaw cycles; rain; ice build-up; older design/construction practice
Terra-cotta block	Crazing (formation of small random cracks in the glaze); spalling (in glaze and material); deterioration of metal anchoring	Weather condition; water; improper handling; older design/construction practice
Wood panels and weatherboarding	Wood deterioration; loose planks	Periodic wet/dry conditions; insect infestation; nails missing/damaged

Aluminum, metal and synthetics panels	Deformation; loose panels; girt beam damage	Age; installation errors; inadequate girt beam design
All-concrete walls	Crack in concrete (diagonal, vertical, horizontal, or spider-web kinds)	Inadequate reinforcement; corrosion of reinforcement; settlement; in-plane or out-of-plane excessive loads; low-quality aggregates; and concrete
Adobe	Cracks	Temperature variation; external loads; especially vibrations
Stucco or render	Plaster damage (peeling or cracked)	Damage supporting metal mesh or lath; age
Exterior insulation and finish system (EIFS)	Cracking; surface wetting; mold; attachment deficiencies; sealant failure	Moisture penetration; impact by foreign objects; inadequate installation; insect infestation
Green walls	Overgrown vegetation; wall damage and aesthetic loss	Lack of proper maintenance; excessive moisture; inappropriate selection of plant
Glass	Crack; distorted frame	Material issues; wind load effect; inadequate installation

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1.5: Building Envelope Engineering Management

Building envelope engineering management focuses on all activities needed to maintain the safety and integrity of the building envelope to ensure it performs its function. These activities broadly include inspections, engineering analysis and evaluations, remediation planning, and reconstruction. Figure 1.5 summarizes these activities and outlines specific tasks related to them.

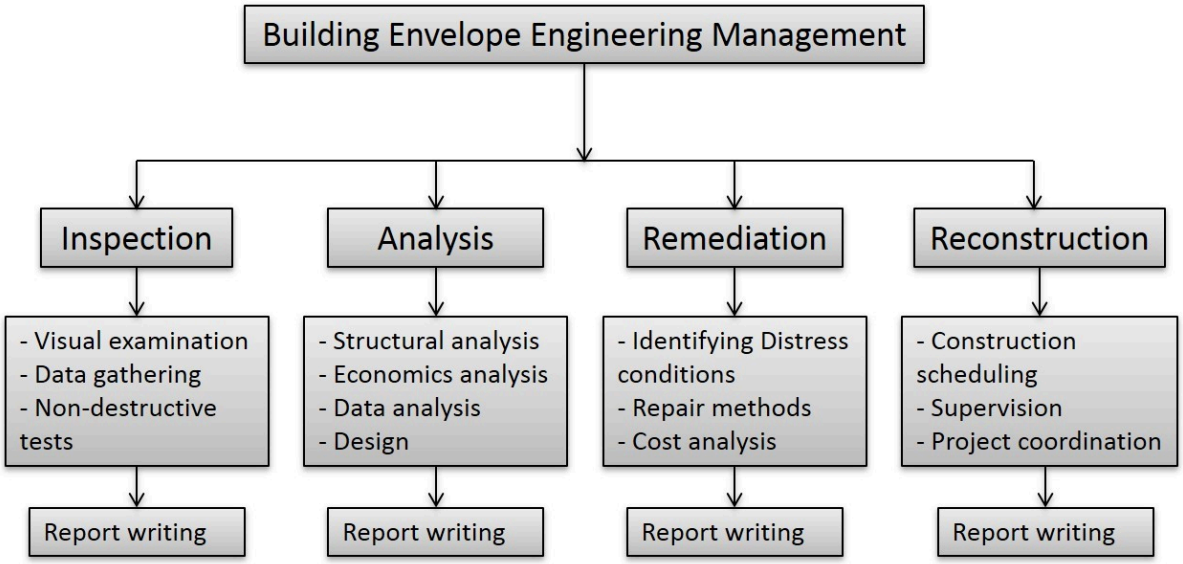


Figure 1.5 Activities in building envelope engineering management

(Chart of building envelope engineering management: inspection, analysis, remediation, and construction, each leading to report writing.)

Inspection activities can range from a cursory review, involving distant observation of façades on all sides of the building, to a complete close examination using scaffolds, with or without non-destructive evaluation (NDE) methods. NDE methods may require monitoring certain distress conditions over an extended period of time, compiling data on specific parameters related to these conditions. For example, vibration monitoring and acceleration data compilation might be conducted for areas on the façade, such as lintels that support masonry above windows, which are known to be susceptible to vibrations causing cracks in masonry walls.

The scope and level of detail of a building inspection are influenced by factors such as the building's type, occupancy, age, history of distress or damage, height, geographic location, and applicable municipal regulations. Inspection requirements – including the type and frequency of assessments – vary across jurisdictions. Typically, a "critical" inspection entails a comprehensive and meticulous evaluation, often involving close examination of significant portions of the façade, sometimes performed from scaffolding or other elevated platforms. Such inspections are generally required in intervals of 3 to 5 years and may also be prompted by severe or urgent conditions that necessitate immediate remediation. In contrast, a "cursory" or "routine" inspection generally involves a less intensive, preliminary visual assessment conducted from a distance on an annual basis, provided the building undergoes regularly scheduled inspections as required by local authorities.

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1.6: Role and Responsibilities of Engineers and Architects in Inspection, Repair and Rehabilitation of Building Envelopes

Regardless of the type of inspection – whether critical or cursory – most municipalities require that it be performed by, or under the direct supervision of, a licensed professional. This professional may be an architect, structural engineer, or an individual with appropriate technical training and proper registration to conduct building envelope inspections. The inspector, as the professional of the record, is responsible for the entire process and must sign off on all related documents and reports. Since municipal requirements regarding who may conduct the inspection can vary, it is essential to review and comply with the specific rules and regulations governing building façade inspections in the relevant jurisdiction.

The responsibilities of the professional of the record extend beyond their own expertise. They need to ensure that all aspects of the building envelope inspection are covered by various types of expertise. Beyond the three main areas of expertise outlined in Figure 1.4, building envelope inspection projects may also require the services of additional experts. Therefore, it is important for the professional of the record to identify the extent and level of external expertise needed to complete the inspection process and include the associated budget in the proposed cost for the entire project. It is important to note that these services are specific to the inspection process and do not involve those needed for fabrication, installation, reconstruction, and repair, as these are primarily the responsibility of the contractor hired for reconstruction and repair. Table 1.3 provides a summary of the expertise that may be needed to support the professional of the record in conducting the inspection process.

To better acquaint the reader with the extent of these services, especially as they pertain to the structural engineering aspects of building envelope inspection, this section provides examples of the analyses and investigations often needed as part of the inspection. Additional information regarding the non-destructive evaluation (NDE) methods that may be required as part of the inspection is discussed later in Chapter 8.

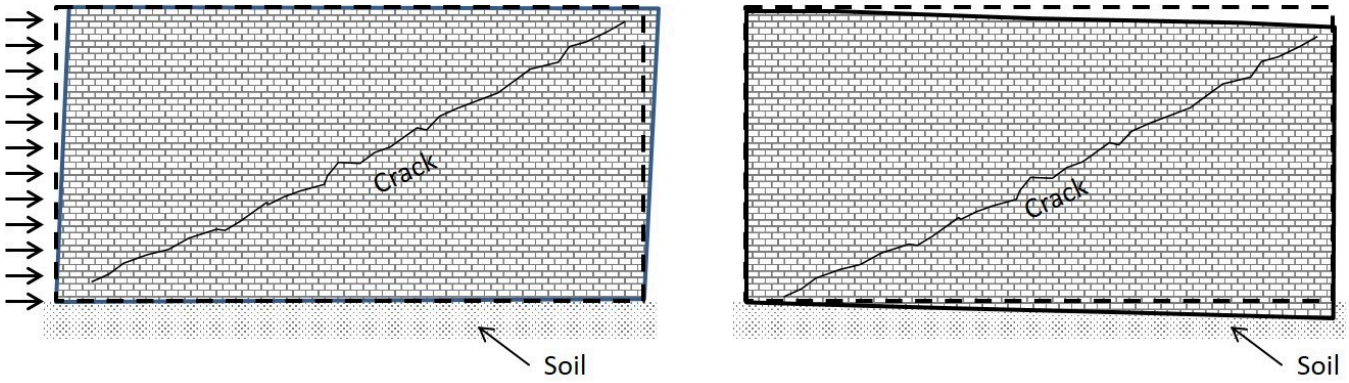
Table 1.3 An overview of external services that may often be needed for building envelope inspection

Expertise	Types of Services
Agriculture and botanic expert	Providing consultation with vegetation in green roof and green wall designs
Asbestos consultant/inspector	Identifying hazardous asbestos situation
Asbestos contractor	Compiling data needed in areas where asbestos is present
Environmental engineer	Compiling data on air samples and molds
Fire protection expert	Providing consultation with potential issues with fire hazard of building envelopes and their effect on the safety of the entire building
Geotechnical engineer	Compiling data on soil properties and soil condition

Glass façade expert	Providing consultation with potential damage scenarios and requirements on detailing
Land surveyor	Preparing plat of survey and other geodetic information as required by the municipality
Laser-scanning surveyor	Providing laser scanning of the building façade before the start of inspection and after the construction is completed (i.e., as built). This is recommended general practice, but it is especially needed for historic buildings
Reserve study expert engineer	Conducting an engineering economics and financial study to determine the capital investment and funds needed to put on reserve for future potential envelope (and other building components) damage
Material testing	Taking samples of materials for identification of mechanical properties
Structural monitoring engineer	Installing proper sensors and compiling data on structural parameters
Structural/mechanical testing engineer	Conducting NDE methods such as petrographic studies, ground penetration radar, thermographic studies

Engineering evaluation and analysis constitute a major portion of the inspection process. These activities require engineering data on the size and types of various façade components and information on the mechanical properties of materials. Depending on the geometry and configuration of the façade, structural elements such as lintels, shelf angles, bond beams, and cantilevers (found in canopies and awnings) may be present, necessitating detailed load calculations. Structural analyses for stress, deformation, and vibration computations are then required. Estimating damage, its growth, and its causes also involve structural analyses by simulating loads and load variations over time.

For example, when investigating the causes of a diagonal crack in a brick or CMU masonry wall, the crack could be attributed to either (a) a severe in-plane lateral load or (b) foundation settlement (see Figure 1.6). While in a visual inspection both scenarios appear to be identical, separate analyses would be necessary to simulate the loading regime and to investigate the real cause, the extent of the damage and the potential for future damage growth. These analyses are necessary so that a correct remedial action can be prescribed as a permanent solution. These analyses require specialties, that are generally outside of the inspector's own area. The professional of the record is responsible for making sure proper structural evaluations are conducted by a qualified engineer.



(a) Crack because of lateral in-plane forces

(b) Crack because of partial soil settlement

Figure 1.6 Diagonal crack in masonry wall

(Two brick walls with diagonal cracks: left from lateral in-plane force, right from foundation settlement, showing similar damage can have different causes.)

When estimating loads, one must consult an appropriate current code, such as ASCE-7 (2022), and obtain information on the densities of materials commonly used in building envelopes. Table 1.4 summarizes information on material densities. The values provided in Table 1.4 are used to estimate the dead load on façades, walls, their foundations, and other structural components. In addition, other types of loads must be considered, including live loads and load effects from wind, earthquakes, snow, ice, and rainwater, which may create a pond-kind of effect on the roof. In green walls, the effects of rain and snow may also impact the walls (in addition to the roof) as vegetation gains weight when wet.

Table 1.4 Material weights[1]

Material	Weight, pounds (kilograms)	Per	Material	Weight, pounds (kilograms)	Per
Aluminum	170 (2,723)	Cubic foot (cubic meter)	Insulation	2.0 (32)	Cubic foot (cubic meter)
Cast iron	450 (7,208)	Cubic foot (cubic meter)	Limestone	171 (2,767)	Cubic foot (cubic meter)
Cement	94 (1,506)	Cubic foot (cubic meter)	Lumber	35 (561)	Cubic foot (cubic meter)
Concrete	150 (2,403)	Cubic foot (cubic meter)	Brick/CMU walls 4-inch thick 8-inch thick 12-inch thick	42 (205) 84 (410) 125 (610)	Square foot (square meter)
Crushed stone	2,500 (1,483)	Cubic yard (cubic meter)			

Glass	161 (2,579)	Cubic foot (cubic meter)			
Gravel	2,700 (1,601)	Cubic yard (cubic meter)			
Gypsum board	50 (804)	Cubic foot (cubic meter)	Steel	490 (7,849)	Cubic foot (cubic meter)
Ice	57 (913)	Cubic foot (cubic meter)	Terra Cotta[2]	70 – 120 (1,120 – 1,922)	Cubic foot (cubic meter)
1 Information mainly taken from ASCE (2022) and FRW (2018).					
2 Lower value is for unfilled and upper value for filled blocks.					

In situations where structural monitoring and data compilation on certain parameters (such as deflections, stress, or accelerations) over an extended period of time are desired, the structural analysis may be more involved and require structural dynamic analyses. This is especially true when the cause of stress or deflection is a vibration source that may cause steady damage to the façades and roof in the form of cracks. Structural dynamic analyses will then be useful in identifying the frequency contents of the input vibration and computing the natural frequency of vibration of an affected component (e.g., a lintel). Appropriate modifications to the component can then be made to mitigate the effects of steady vibrations.

Structural design often becomes involved when the replacement of portions or entire walls, or components such as lintels, shelf angles, anchors, and foundations, is necessary. Additionally, there are other structural members that are not necessarily part of the building envelope but use the envelope for support. Examples of these structures include porches, canopies, awnings, rooftop water tanks, flag poles, ornamental pieces, stairs, external fire escapes, and chimneys. In this book, these are referred to as ancillary structures. Although not considered directly as parts of the building envelope, ancillary structures are still within the responsibilities of the architect or engineer performing the envelope inspection. It is imperative that the professional of the record note any deficiencies in these ancillary structures and the specific issues they may impose on the envelope, and bring these to the owner's attention. The owner needs to know that the evaluation and design of these systems will require additional manpower, and these activities should be clearly outlined in the contract so that all parties understand the scope of work required for ancillary structures.

Another responsibility of the professional of the record is the preparation of inspection findings. These findings, along with a remediation plan to address damage to the building envelope, are presented to the owner in the form of a report. This report provides detailed information on the types of distress conditions, plausible causes, repair methods, materials to be used, costs, and the expected lifespan of the repairs. When more than one alternative is identified for any repair or reconstruction, it is important that the report includes an economic evaluation of these alternatives, considering the short-, mid-, and long-term benefits and costs. This is further explained in Section 1.8 of this chapter and in more detail in Chapter 9 of this book.

A complete set of drawings generally accompanies the final report. These drawings are stamped by the professional of the record and prepared in a format acceptable to the municipal authority for issuing the construction permit. The report and accompanying drawings must be clear and detailed enough to be readily used by the contractor hired to perform the reconstruction.

The building envelope repair and reconstruction process often differs from new construction. Therefore, it is imperative that the professional of the record remains actively involved throughout the process. As the contractor begins reconstruction, new information may become available, necessitating changes in the design of structural components, repair methods, or the overall direction of the reconstruction process. Effective communication among all parties – the contractor, owner, and the professional

of the record – is crucial to avoid mistakes during construction. This phase of work requires additional tasks from the professional of record, which should be clearly indicated and outlined in the original contract with the owner to avoid misunderstandings or confusion among all parties involved in the reconstruction and repair project.

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1.7: Quality Assurance and Professional Ethics

In addition to the responsibilities discussed in Section 1.5, the professional of the record is also accountable for ensuring that the safety, health, and welfare of the public are upheld. This includes conducting all activities related to the condition assessment of building envelopes in accordance with established standards of practice and professional quality. Consistent with the fundamental principles of ethical conduct, the professional of record is expected to:

- Recognize the problem, use their knowledge in their area of expertise, and seek professional services in other areas as needed (as explained in Section 1.5).
- Prepare quality documentation that is accurate, clear, and thoroughly outlines the procedures or methods used.
- Exercise appropriate judgment based on sound analysis and design rather than merely complying with external pressures.
- Establish a quality assurance (QA) process to ensure the accuracy and quality of all analyses and calculations, including those conducted for safety and design.
- Follow the code of ethical conduct prescribed by the professional society within the professional of the record's area of expertise.

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1.8: Capital Development and Investment Returns on Building Envelope Repair and Maintenance

Depending on the condition of the building envelope, there may be multiple options for recommending remedial actions to the owner. When conditions are critical and pose a safety risk, immediate remedial measures must be implemented. These may apply to the entire building envelope or to specific portions of it. In less urgent cases – where deferred maintenance is a viable option – it is essential for the owner to receive an economic study that outlines and compares alternatives such as:

- Taking no action.
- Performing immediate repairs
- Planning for repair or replacement at a later time.

Each option carries its own set of costs and benefits. For example, choosing to do nothing may lead to significantly higher costs in the future and could ultimately prove detrimental. On the other hand, postponing repairs may give the owner time to allocate or accumulate the necessary funds.

Economic studies play a critical role in assessing the return on investment and supporting long-term capital planning for building envelope systems. Such analyses enable owners to make informed, data-driven decisions regarding repair, replacement, and maintenance strategies. In addition to economic evaluations, a reserve study serves as an essential financial planning tool for estimating the funds that should be allocated and set aside to cover anticipated and unforeseen future expenditures. In many jurisdictions, reserve studies are not merely recommended but mandated by state or local regulations, particularly for condominium and homeowners' associations (HOAs). These governing entities are typically required to ensure that reserve studies are conducted at regular intervals – commonly every three to five years – to maintain compliance and ensure the continued financial stability of the property. A properly conducted reserve study provides a forward-looking framework for sustainable asset management, helping to avoid financial shortfalls, unexpected special assessments, or deferred maintenance issues that could compromise building performance and safety. The principles and methodologies underlying such studies, along with their practical applications in façade management and capital planning, are examined in greater detail in Chapter 9 of this book.

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1.9: Common Terminologies Used In Building Envelope Inspection

Knowing the terminologies commonly used in building envelope inspection is important to avoid any confusions and miscommunications with the contractor. The following presents a list of these terminologies.

Anchor: Bolt and nut system used to attach shelf angles or brackets to masonry.

Afar examination: Inspection conducted using binoculars.

Annual examination: An inspection conducted every year, see also “cursory” examination.

Awning: A covering attached to the exterior wall of a building, typically above windows, doors, or over patios and decks.

Backer rod: A flexible rope-like material used as fillers between joints.

Brick and CMU positions: As illustrated in Figure 1.7, these positions are referred to as stretcher, soldier, header, stretcher rowlock, sailor and rowlock. Stretcher units are laid horizontally with their long sides facing outward. Soldier units are placed vertically with their long, narrow sides facing outward. Header units are laid horizontally with their short ends facing outward. Rowlock stretcher units are laid on their narrow sides with their long edges facing outward. Sailor units are placed vertically with their wide faces facing outward. Rowlock units are laid on their narrow sides with their short ends facing outward.

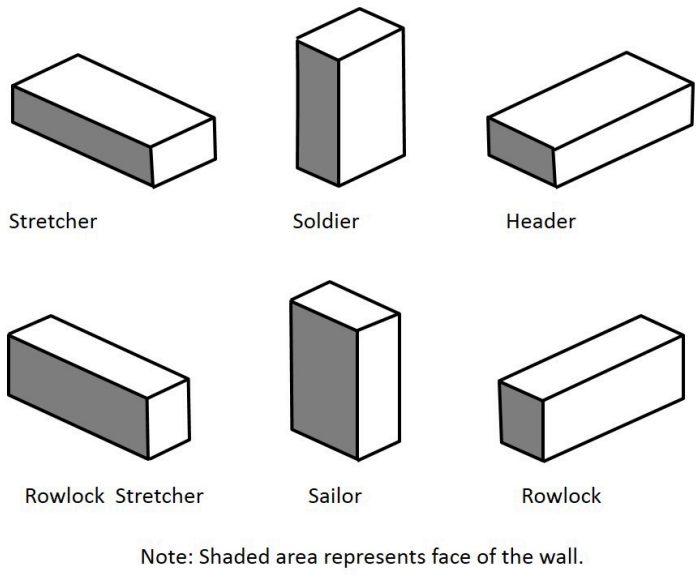


Figure 1.7 Brick and CMU position (Adopted from BIA, 2009)

(Diagram of masonry unit positions: stretcher, soldier, header, rowlock stretcher, sailor, and rowlock, showing horizontal or vertical orientations.)

Bond beam: A beam made up reinforced concrete, or reinforced masonry blocks, that is built on masonry walls to provide support for floor joists (see Figure 1.8)

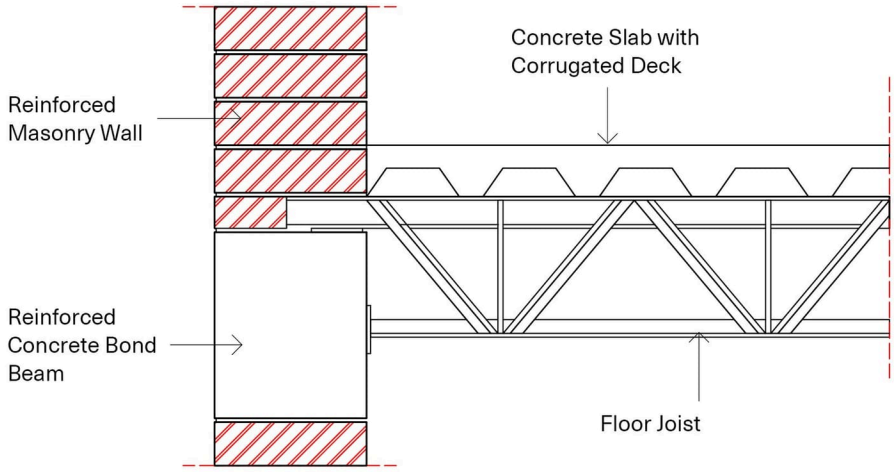


Figure 1.8 An example of a reinforced concrete bond beam

(Cross section of a masonry wall and a concrete beam (called bond beam) embedded in the wall: the bond beam supports floor beams.)

Building corner: For façade inspection purposes, it is at the intersection of two walls.

Canopy: A projecting overhead covering that provides protection, often built over entrances, windows, or walkways.

Caulk: A low- or intermediate-performance compound used as a sealer mainly in interior applications. Note: see definition for "sealant," which is a high-performance compound for exterior façade applications.

Categories I, II, III and IV buildings: These vary across municipalities and generally refer to the building types in terms of the occupancy (for façade inspection and safety purposes).

Change order: A request to change a detail that is different from the one on the original drawing.

Cladding: The material and/or element used as the finish on the exterior of the façade.

Control joint: A cut made into concrete slabs to allow expansion and contraction in concrete – Note: this is different from an expansion joint.

Close examination: An inspection that requires an architect or engineer to use scaffolds and conduct a close evaluation of the façade condition.

CMU: Concrete masonry unit.

Coping stone: A piece of stone at the top of a parapet wall (see Figure 1.9). The types are plain straight, twice weathered and once weathered.

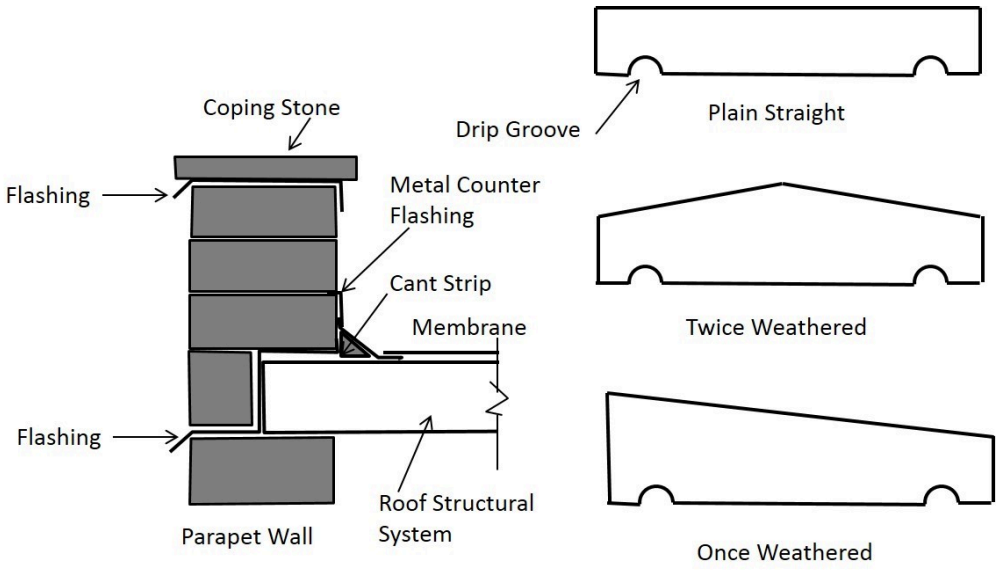


Figure 1.9 Types of coping stone

(Parapet wall cross section with coping stone: coping stones types are plain straight with flat top, twice weathered with two slopes and once weathered with one slope.)

Course of brick: One layer of brick in horizontal direction.

Crack meter: A device used to monitor the width and growth of cracks in masonry. Simple ones are made up of two pieces with markings. Each piece is mounted on one side of the crack. The two pieces slide over each other providing information on crack growth.

Critical examination: A façade examination that requires close observation of the wall condition using scaffolds and covering the required façade areas per recommendations by the municipality.

Cornice: A decorative molding or projection that typically runs along the top of a building constructed just below the roofline.

Cursory examination: A façade examination intended for condition assessment between critical examinations. This inspection is typically performed on an annual basis and involves a general visual observation of the façade condition from a distance.

Door terminologies: As illustrated in Figure 1.10, terminologies pertinent to doors are jamb (an upright of frame on which door is hanging); rabbet (groove or recess at the edge); and threshold. Doors are often marked with ratings as one-hour, two-hour, etc., which refer to fire protection.

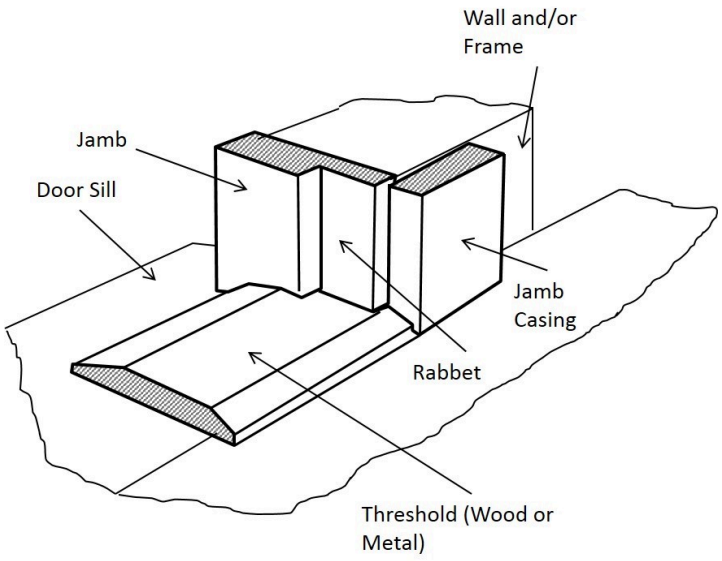


Figure 1.10 Terminologies used in doors

(Door terminologies: door sill, jamb, jamb casing, rabbet and threshold are shown.)

Drone: An unmanned aircraft system (UAS) designed to capture detailed visual data of a building's exterior, including hard-to-reach areas, to assess the envelope integrity.

Drop: A vertical area of façade designated for inspection when using moveable scaffolds.

Due-diligence inspection: An inspection conducted before sale of the property.

Efflorescence: A crystalline deposit on surfaces of masonry walls.

EIFS: External Insulation Finish System, also known as synthetic stucco

Envelope: A term used to generally refer to the building enclosure including the roof.

Expansion joint: A vertical gap in walls to allow wall expansion and contraction – note: this is different from a control joint.

Exploratory opening: A façade element (such as a brick) removed for inspection of the condition behind the wall.

Façade: A term used to generally refer to the building enclosure excluding the roof.

Façade inspection: Evaluation of the building enclosure for safety purposes as mandated by a local jurisdiction.

Flashing: A metal or neoprene sheet used to wash water out at areas above windows and doors, under the coping stone, and between wall and foundation (see for example Fig. 1.9).

Girt: A horizontal beam, usually installed in the mid-height between two floor levels, to help reduce the bending of the cladding in the vertical direction and protect it from damage due to wind pressure.

Inspection: See façade inspection.

Lintel: A beam at the top of the window or door frame or other openings.

Ledger beam: A timber beam, usually with a nominal thickness of 2 inches (51 mm) secured to the wall via anchor bolts to provide support for hangers used in timber joists (see Figure 1.11).

Laser imaging, detection, and ranging (LIDAR): A laser-based method used for distance measurement to identify surface shape of walls and other building components.

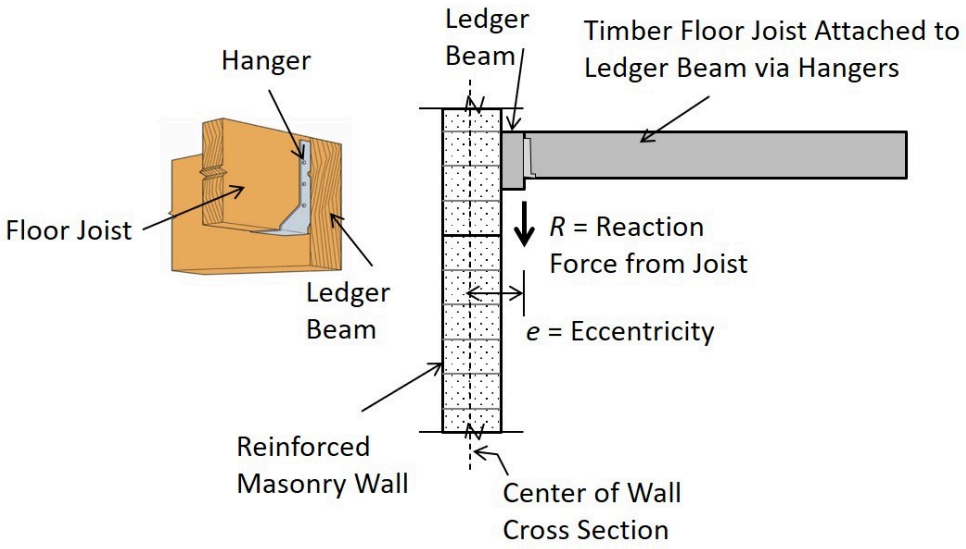


Figure 1.11 Details of a ledger beam

(Ledger beam installed on a wall: ledger beams provide a flat surface for mounting joist-hangers supporting timber beams.)

Maintenance: Periodic inspections to make sure the building envelope is safe. This may or may not involve repair.

Mortar: A cement-based material used in between masonry blocks.

NDE method: A non-destructive, non-invasive evaluation method.

NDT method: A non-destructive, non-invasive test method.

Opening: A façade element (such as a brick) removed for inspection of the condition behind the wall. See also exploratory opening.

Parapet wall: A short wall extended above the roof (see Figure 1.9).

Professional of the record: An engineer or architect in charge of façade examination.

Porch: An open structure made up of posts and beams attached to the main building, usually on the back. Porches are mainly made up of wood or steel and contain external stairs and decks.

Reconstruction: A portion or entire building envelope rebuilt to its original condition.

Rehabilitation: A series of major replacements or changes in a portion or the entire envelope to maintain the intended function.

Render: See stucco.

Repair: A series of minor replacements or changes in a portion or the entire envelope to maintain the intended function.

Reserve study: An engineering and financial analysis conducted to determine the capital investment and funds needed to set aside for the repair, rehabilitation, and replacement of major components in multi-occupancy buildings. This study is often required by certain jurisdictions to ensure that adequate reserves are maintained for future maintenance and capital expenditures.

Retrofit: A series of changes that would improve the function of a portion or entire envelope to bring it to a higher level to meet new demands.

Scaffold: A moveable or stationary temporary structure used for façade repair (see Figure 1.12).

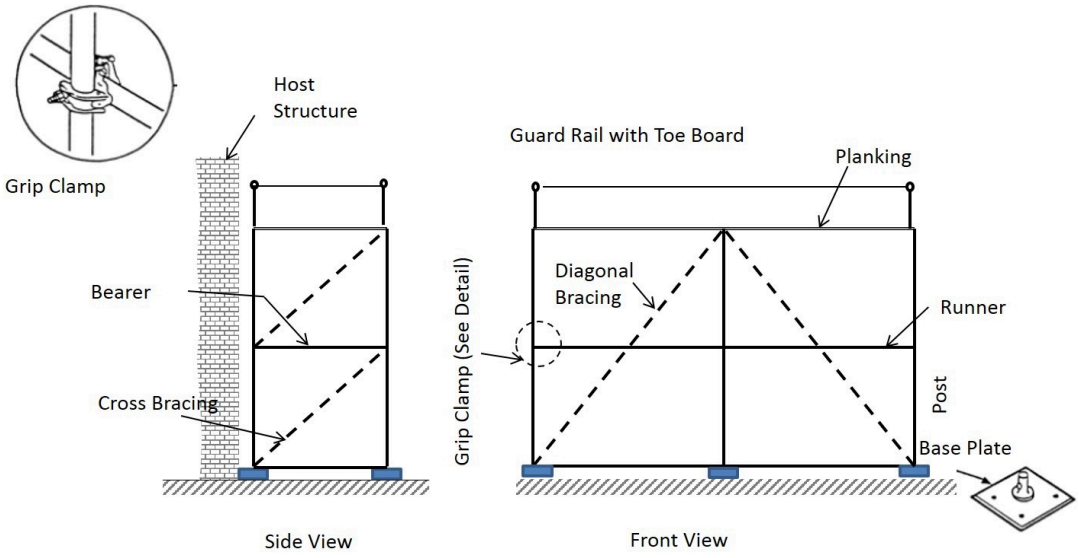


Figure 1.12 An example of a stationary scaffold (adapted from Mohammadi, et al, 2015)

(Two-story scaffold: this is made up of steel tubular structural elements connected with grip clamps and diagonal bracings for stability.)

Sealant: A compound (often high-performance) used for water proofing in exterior façade applications. Note: see the definition for "caulk," which is a lower-performance compound usually for interior applications.

Shelf angle: An angle attached to the structure to support a veneer wall.

Spandrel beam: A beam that runs around the edge of each floor and is integrated with the floor slab.

Stairs: Terminologies used for stairs are illustrated in Figure 1.13 and are "going," "nosing," "rise," and "tread." A landing is a small area when the stair turns. The riser refers to the vertical part, tread refers to the horizontal part, nosing is the edge, and going refers to the width of the tread. The rise refers to the height of the step.

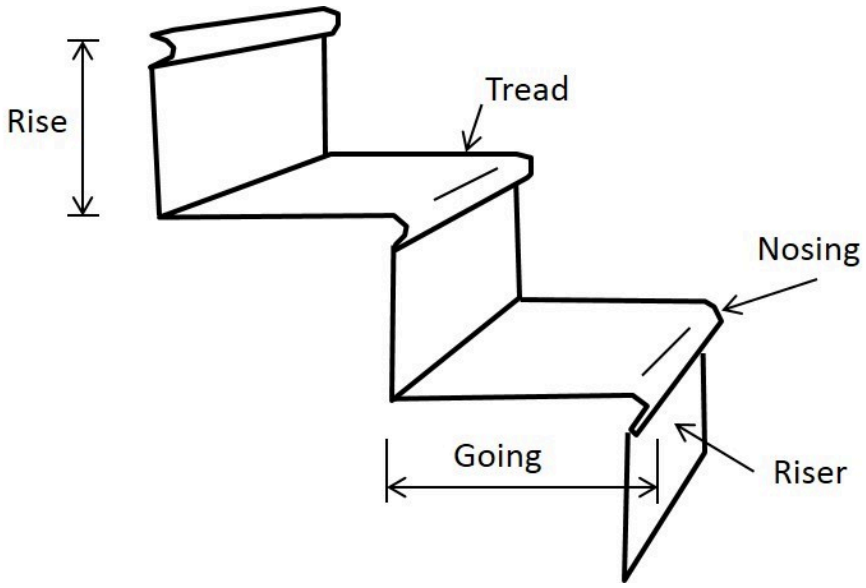


Figure 1.13 Stair Terminologies

(Stair terminologies: riser, tread, nosing and going.)

Steel relief angle: Same as shelf angle.

Stucco: A plaster-like exterior finish applied in multiple layers over a substrate such as metal lath, wire mesh, or masonry in façades.

Swing stage: An area where moveable scaffold is installed for façade inspection.

Terra-cotta block: A plain or ornamental block made up of baked clay attached to the structure through metal anchors (see Figure 1.14).

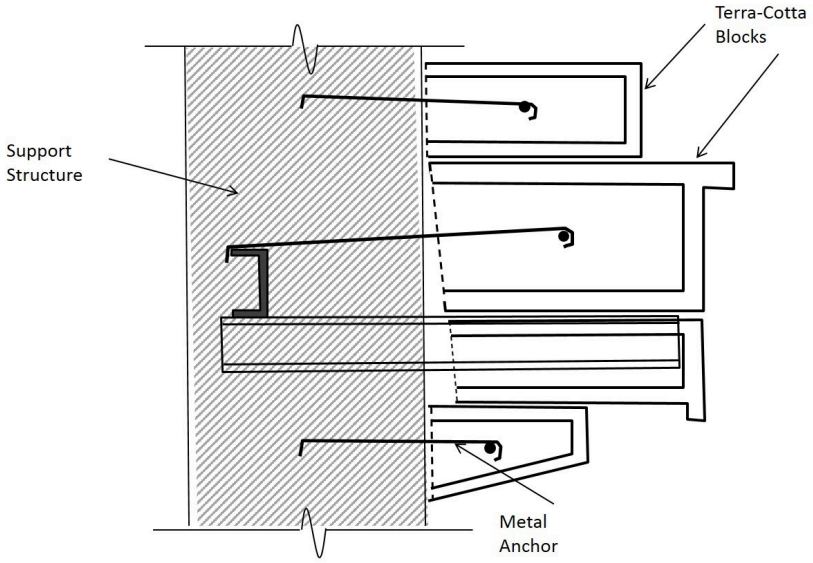


Figure 1.14 An example of a terra cotta block, adapted from Mohammadi and Modares (2017)

(Terra-cotta blocks: block are attached to a support structural system using metal anchors.)

Thermal break: A component or material with low thermal conductivity used to reduce heat transfer and improve energy efficiency.

Tilt-up panels: Reinforced concrete panels cast horizontally and then lifted into place to form the building façade.

Tuckpointing: A repair done on mortar between masonry units.

UAS: Unmanned aircraft system. Also. see the definition for “drone.”

URM: Unreinforced masonry.

Walls: Different types of walls are curtain, load bearing, parapet, veneer, tilt-up, and green walls.

Water test: A test by splashing water over façade usually for about 30 minutes to detect leaks.

Weep hole: A small opening, usually at the mortar joint, to allow moisture out.

Wick: A rope used in weep holes to drain condensation water and moisture out.

Whiskers: See efflorescence.

Window sill: A horizontal piece of stone or other masonry used below a window frame.

Wythe: A continuous vertical section of a brick or masonry that has one unit in thickness. Note that a course runs horizontally along the wall, while wythe is vertical.

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1.10: Chapter References

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1.11: Exercises

E1.1 A 4-inch (102-millimeter) thick, 25-foot-tall (7.6-meter) brick wall is supported by a continuous foundation. Compute the maximum compressive stress, using pounds per square inch as the unit (or megapascal) on the mortar because of the weight of the wall. Note: The effective thickness of this wall is about 3-8th of an inch (10 millimeters) less than the nominal thickness.

E1.2 Following most local codes involving façade inspection requirements, the professional of the record is

a) The building owner.

b) The contractor who does the repair.

c) The engineer or architect who conducts the inspection and prepares the report.

d) The property manager.

E1.3 An inspection report refers to what is called a “swing stage.” What is this report referring to?

a) Some area in the interior of a commercial building.

b) Some portions of a façade not accessible.

c) An area where moveable scaffold is installed for façade inspection.

d) A façade that has already inspected.

E1.4 Which of the following statements is most accurate?

a) Wythe is a term used in brick work referring to a horizontal row.

b) Course is a term used in brick work referring to a vertical column.

c) Both wythe and course mean the same thing in a brick or other masonry wall.

d) Course is one layer of brick in horizontal direction; while wythe is a vertical section with one unit in thickness.

E1.5 Which of the following statements is *not* accurate?

a) A control joint is generally referred to a cut made into concrete slabs to allow expansion and contraction.

b) An expansion joint is a vertical gap to allow wall expansion and contraction.

c) Steel relief angle is another term used for a shelf angle.

d) A parapet wall is a temporary wall built when repair is being done.

E1.6 While conducting an inspection of brick façades of a building, you noticed severe rust, distorted elements and unstable connections in a steel fire escape in the east façade. Which of the following actions is considered most appropriate on your part?

a) I will write a section in the report identify the situation as an observed area of safety concern that needs to be addressed.

b) I will conduct a complete investigation of the condition of the fire escape and make it a main part of my report.

c) I will ignore the situation, since it was not written in my contract.

d) I will make a brief reference to it in my report without any elaborations.

E1.7 Which of the following statements is *inaccurate*?

a) A core sampling to determine the strength of concrete in a wall is an example of NDT methods.

b) Measuring the changes in the width of a crack in a wall over several months is an example of long-term monitoring.

c) Conducting NDT/NDE methods and monitoring often require hiring appropriate expertise.

d) NDT/NDE methods and/or long-term monitoring are always an integral part of the façade inspection contract whether or not mentioned in the contract.

E1.8 A structural dynamic analysis may become necessary as part of a building inspection project when

a) Damage because of vibration of structural components is suspected.

b) Information on the frequency contents of structural members such as lintels is needed.

c) Operation of mechanical systems is suspected of causing resonance.

d) All of the above.

E1.9 Which of the following best describes the purpose of an engineering economics study as part of a façade inspection and evaluation?

a) To indicate the increase in the cost of property after any repair.

b) To offer the owner with information on return on the investment and a comparison of various options including “do nothing.”

c) To provide the owner with an idea on the cost of items not within the contract.

d) To offer the owner with costs associated with such items as replacing windows and doors.

E1.10 Review the situations (i), (ii) and (iii) listed below. Then decide which kind of expertise would be needed to best address them.

Situation (i) foundation settlement is suspected in causing cracks in a wall.

Situation (ii) a portion of a façade is made up of reinforced concrete and it has signs of concrete spalling and patchy cracks.

Situation (iii) the façade inspection reveals severe wetness and potential for mold growth on a wall.

a) A structural engineer will be needed for all of these situations.

b) A geotechnical engineer for (i), concrete and material specialist for (ii) and structural engineer for (iii).

c) A geotechnical engineer for (i) and (ii) and an environmental engineer for (iii).

d) A geotechnical engineer for (i), material and concrete specialist for (ii) and an environmental engineer for (iii).

E1.11 A façade inspection report refers to the term “weatherboarding” in portions of the façade. What exactly the report is referring to?

a) Portions of the façade have been repaired by a different material than the original one.

b) Portions of the façade are made up of wood panels.

c) The façade is not accessible for inspection.

d) Portions of the façade is made up of metal cladding.

E1.12 What kind of façade structural member is a “girt?”

a) A beam above a window or door opening.

b) A beam intended to sustain wind load and reduce the bending of claddings.

c) A spandrel beam on perimeter of a building.

d) An additional column for shelf angle support.

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2: BRICK WALLS

Chapter Synopsis

This chapter covers brick walls (primarily in the form of veneers), connection of brick veneer to the structural system, kinds of potential damage, structural and architectural damage issues, and repair methods. Illustrative examples pertinent to materials presented in this chapter are offered in Appendix A.

2.1: Introductory Remarks

2.2: Brick Veneer Walls and their Details

2.3: Use of Shelf angles in Veneer Walls

2.4: Requirements for Veneer Design in Seismic areas

2.5: Design Requirements for Anchored Veneers in Areas Subject to High Wind

2.6: Flashing, Weep Holes and Expansion Joints

2.7: Damage to Brick Walls and Repair Methods

2.8: Chapter References

2.9: Exercises

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2.1: Introductory Remarks

Brick walls are among the most common façades in low to medium rise buildings. They are primarily in the form of veneer and load bearing walls. Load bearing walls may be secondary (backup) walls or can also be used as façades. The popularity of brick walls is mainly because of their durability, relatively low cost and architectural appeal. In this chapter, we introduce

- The structure of brick walls and their components including their support systems.
- Areas where proper architectural detailing will need to be considered. This includes, for example, proper design for flashing.
- Veneer wall design requirements including its relation with supporting elements such as shelf angles and foundations.
- Damage potentials and methods of repair to address damage areas.
- Problems and shortcomings that need to be corrected to improve the performance and durability of brick walls.

The coverage of the materials in this chapter is intended to familiarize the reader with common areas that one must note, and attend to, when conducting an inspection of a brick wall. There are, however, situations where damage scenarios may be unique and not necessarily considered as “common problems.” In these situations, more thorough investigations with the aid of non-destructive tests may become necessary; and a program of monitoring may have to be considered to pinpoint to the source of problems and damage situations.

In addition to materials presented in this chapter, and as additional resources, the reader is also referred to ACI 355.2 (2007), ACI 355.4 (2011), BIA 91987), BIA (1986) and TMS (2016).

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2.2: Brick Veneer Walls and their Details

Veneer walls form the building's façade and must be properly supported. In low-rise buildings - typically those with a height of 30 feet (9.15 meters) or less - a brick veneer wall may rest directly on the foundation system. However, when the building height exceeds 30 feet, the veneer wall must be supported by the structural system using elements known as shelf angles. In such cases, both the foundation and the shelf angles contribute to supporting the weight of the veneer wall. Additionally, for lateral stability, the veneer must be securely anchored to the backup wall or the structural system. The design of these anchors can vary depending on factors such as building type, height, construction methods, and structural system.

Structural analysis and design considerations for veneer walls include:

- The size of the footing to support the weight of the wall. This is determined based on the soil bearing capacity and the weight of the wall.
- Selection of appropriate sizes for shelf angles and lintels. The determination of the size of a shelf angle and design for the anchors or welds, for the attachment of the angle to the structure, are achieved by using methods of structural analysis and design and by consideration for the weight of the wall. The weight is computed based on the thickness of the wall and its height. A typical thickness is about 4 inches (102 millimeters) - although higher thicknesses may also be used.

Many types of details for the veneer wall and its attachment to the back-up wall or the structural system are available depending on the type of building, the height and construction. In general, no matter what type is used, the brick veneer must be (a) stable; and (b) have adequate support to the back-up wall or to the structural system. The back-up wall may be a load-bearing wall or plywood sheathing and a wood or steel stud frame system. In most low-rise buildings (such as in most residential buildings), the back-up wall is made up of wood studs and sheathing. Wood studs are usually 2 by 4 inches (51 by 102 millimeters) in cross sections and are spaced at every 16 inches (0.41 meters). In certain applications, and especially in older buildings, the back-up wall may also be made-up of bricks, or walls made up of concrete masonry units, which are generally the load-bearing structure of the building. In modern applications, in cases where a lead-bearing back-up wall is used, this wall is made up of concrete masonry units; and the wall is reinforced both horizontally and vertically. The design and detail for the reinforcement follows the procedures recommended in masonry codes, e.g., MSJC (2011).

In highrise applications, the structural frame system, whether made up of reinforced concrete, steel or wood, provides the back-up support for the veneer wall and its components.

While performing a façade inspection, it is important to know the type of veneer wall and its support system. Knowing common details for veneer wall is also important when a remedial plan for wall repair is prescribed. Figure 2.1 illustrates a common detail often used in low-rise applications with the building height in the order of 30 feet (9.15 meters) or less. For additional information on brick veneer details and their designs, BIA (2005), WSCPA (2004), WSCPA (2011) and MSJC (2011) are recommended as additional reference sources.

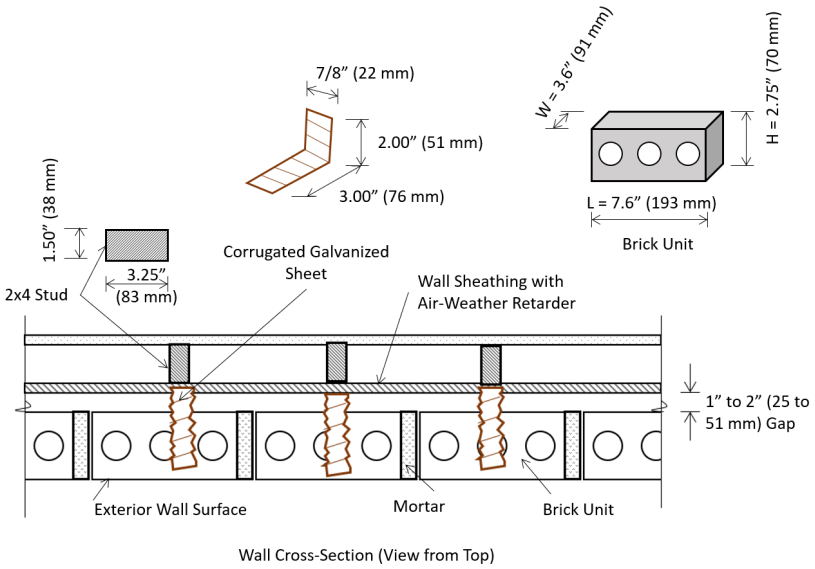
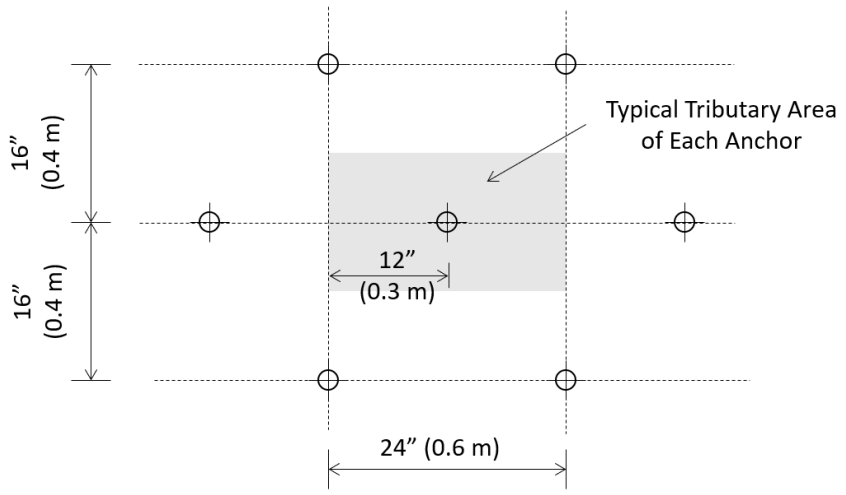


Figure 2.1 Detail of a veneer wall in low-rise buildings with galvanized corrugated sheets used as anchors

(The cross-section of a veneer brick wall is shown. Galvanized corrugated sheets of 7-8th of an inch (22 millimeters) width are used as anchors to prevent wall movement. The figure shows wood studs behind the wall and also the detail of the drywall attached to the studs with 1 inch, 25 millimeters, cavity.)

Notice that in the detail shown in Figure 2.1, the back-up wall is made-up of sheathing with a wood stud frame system. Corrugated (L-shaped) galvanized sheets are used as anchor pieces to maintain the stability of the brick veneer. As stated in WSCPA (2011), these anchors will need to be at least 7-8th of an inch (22 mm) wide, and a base metal thickness of at least 0.03 inches (0.8 millimeters). The corrugation is recommended to have a wavelength of 0.3 to 0.5 inches (8 to 13 millimeters) with an amplitude equal to 0.06 to 0.1 inches (1.5 to 2.5 millimeters). The outstanding legs of the corrugated sheets are embedded in the mortar joints. Working in friction, the anchor pieces prevent the brick veneer from moving outward.

The number of anchors needed is about one per every 2.67 square feet (0.25 square meters) of the veneer surface area. Thus, anchor pieces are generally arranged at distances of about 24 inches (0.6 meters) horizontally and 16 inches (0.4 meters) vertically in a zigzag manner as illustrated in Figure 2.2. A 24-inch horizontal spacing usually translates into anchors being installed where the studs are located, when studs are 2 feet apart. In any case, the maximum recommended spacing is 32 inches (0.82 meters) horizontally and 25 (0.64 meters) inches vertically, see for example TMS (2016). In situations when studs are 16 inches (0.4 meters) apart, anchors are installed at every other stud location. Thus, in the horizontal direction, the spacing between anchors will be 32 inches (0.82 meters), which complies with the maximum limit.

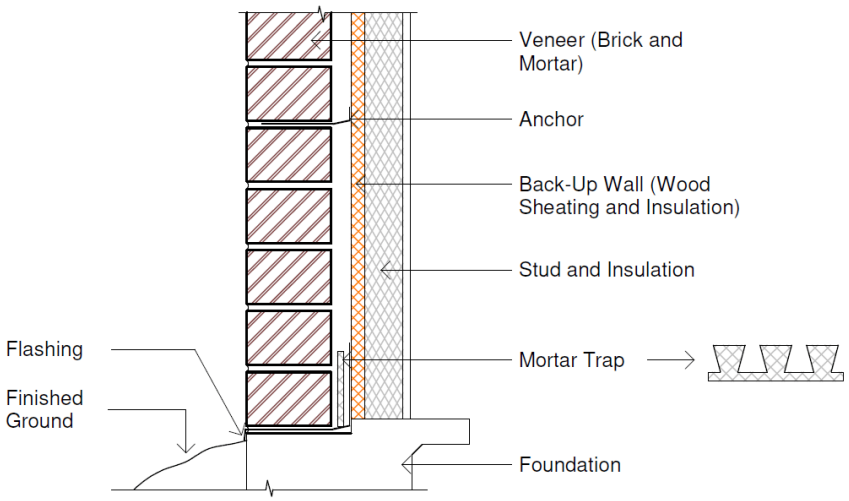


Circles are anchor locations on sheathing

Figure 2.2 Location and arrangement of galvanized corrugated sheets used as anchors

(This figure shows small circles to mark anchor locations on the sheathing in three rows, two at the top row, three in middle row and two in bottom row. The figure shows 16 inches (4 tenth of a meter) vertical distance between anchor locations and 12 inches (3 tenth of a meter) horizontal distance between anchor locations. The tributary area of each anchor is shown as a shaded area).

The veneer support in low-rise applications is illustrated in Figure 2.3. The footing is continuous and is made up of reinforced concrete with a minimum compressive strength of 3,000 pounds per square inch (20.7 megapascals). The bottom of the foundation will need to be sufficiently below ground to avoid freeze line on firm soil with adequate bearing capacity to support the veneer weight.



Vertical Cross Section of Veneer Supported by Foundation
(Note: the Depression in Foundation is Called "Brick Ledge")

Figure 2.3 Detail of veneer on foundation for a building height < 30 feet (9.15 meters)

(This figure shows the cross-section of a brick wall resting on a foundation. Locations of flashing, anchor to prevent wall movement, mortar trap and the backup stud and insulation are shown. The detail of the mortar trap is shown to the right.)

The computation of the pressure on the veneer wall footprint is straightforward and is based on the wall density. This is illustrated in Example 2.1 of Appendix A. Since the backup wall also rests on the footing, its weight and any roof and/or floor load it carries

will also need to be computed and added to the weight of the veneer for designing the size of the footing. The weight is often computed for the unit length of the footing. The stress inserted on the mortar and concrete will need to be computed and checked against the compressive strength of the mortar and the footing concrete.

Table 2.1 summarizes the ultimate and the allowable compressive strength of common types of mortars used in brick veneer wall construction.

Table 2.1 Mortar compressive capacities

Mortar Type	Max compressive capacity, pounds per square inch (megapascal)	Allowable capacity 0.45 times (maximum compressive capacity), pounds per square inch (megapascal)
N	750 (5.17)	337.5 (2.33)
S	1,800 (12.41)	810 (5.58)
M	2,500 (17.24)	1,125 (7.67)

A proper detail (as shown in Figure 2.3) must also prevent the build-up of moisture behind the veneer wall. This is achieved by providing “flashing” and “weep holes,” as explained later in this chapter. The element indicated as “mortar trap” is used to prevent mortar build-up, during construction, which may clog the openings that are provided to keep the moisture out. Mortar traps are generally made-up of high-density polyethylene (HDPE) strands woven into an open mesh.

When checking the strength of the foundation concrete and the mortar against the applied stress, either the Load and Resistance Factor Design (LRFD) or the Allowable Stress Design (ASD) is used. In the LRFD method, the governing design equation is

$$L_u \leq \phi R_n \tag{2.2.1}$$

In which L_u is the factored applied load effect (for example applied stress), which equals the sum of all pertinent load effects, each multiplied by a load actor. If the dead load alone acts, the load factor is 1.4. Then $L_u = 1.4W/A$, where W is the weight of the wall, and A is the area, which is computed based on the effective wall thickness T_E. If the nominal wall thickness is T, then T_E is equal to T minus 3-8th of an inch (9.5 millimeters). With the effective wall thickness being T_E, then $A = 12T_E$ (using inches for T_E), because 1 foot of the length of the wall is considered in computation of the stress. If both dead load and live load apply, the dead load factor is 1.2 and the live load factor is 1.6. The parameter ϕ is the resistance (or capacity) reduction factor. For direct compression on concrete or mortar, it is 0.65. In Equation 2.2.1, R_n is the nominal resistance of mortar or concrete (for example $0.85f'_c$ for concrete).

In the ASD method, the governing design equation is

$$L_a \leq R_a \tag{2.2.2}$$

Where, L_a is the applied load effect (for example stress) without any load factor. Thus, if only dead load applies, $L_a = W/A$. In Equation 2.2.2, R_a is the allowable resistance. For example, for concrete is $0.45f'_c$. For mortar, the allowable resistance values are listed in Table 2.1.

The design of the foundation is done for (a) the size (i.e., the width of the foundation and its thickness); and (b) the reinforcement. As indicated earlier, the size is determined based on the allowable soil bearing capacity. This is a value that must be determined by tests and based on the soil mechanics methods. Typical values for low-rise building applications are usually about 1,500 to 3,000 pounds per square foot (71.9 to 143.6 kilopascals).

In selecting the size of the footing, we usually do not use load factors. Therefore, if the total load on the foundation considering the weight of the veneer, the weight of the back-up wall and the self-weight of the foundation is W_T, then we select a value for the width and thickness of the foundation (say b and h) and check the following inequality to make sure the foundation is acceptable.

$$W_T/b \leq R_b \tag{2.2.3}$$

where,

$$W_T = W + W_B + W_F \tag{2.2.4}$$

W = load from weight of the veneer wall in pounds per foot (kilonewtons per meter) length of the wall

W_B =Load from weight of the back-up wall in pounds per foot (kilonewtons per meter) length of the wall

W_F = load from weight of the foundation in pounds per foot (kilonewtons per meter) of length of the foundation =$b \times h \times \gamma_c$

γ_c = density of concrete = 150 pounds per cubic foot (2,403 kilograms per cubic meter)

R_b = allowable bearing capacity of the soil in pounds per square foot (kilopascals)

The design of reinforcement for the foundation follows the recommendations of the American Concrete Institute (ACI), see for example ACI (2014). If calculations show that no reinforcement for strength is required, a minimum amount of steel will need to be provided per ACI.

When a better adjustment of bricks at the time of construction is required, a 2-piece anchor (adjustable anchor) is used as demonstrated in Figure 2.4. This type of anchor is also preferred in taller buildings. The anchor arrangement follows the same pattern as shown in Figure 2.2.

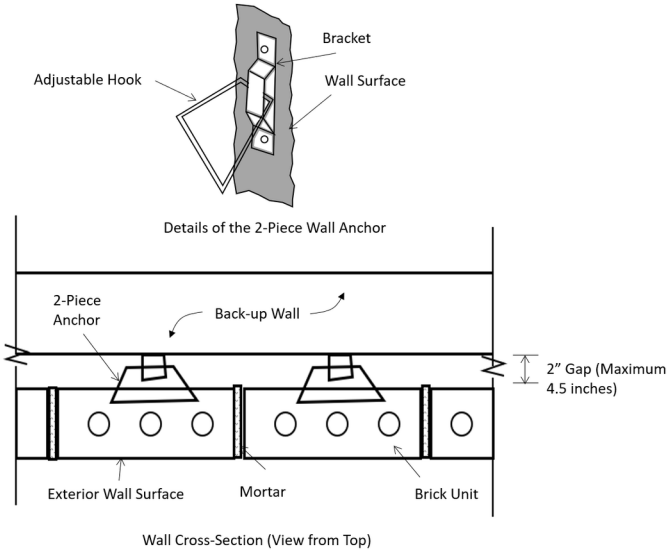


Fig. 2.4 Detail of two-piece anchors

(This figure shows the cross section of a brick veneer wall looking from the top. Two-piece metal anchors to prevent wall movement are shown. To the right, the detail of a two-piece metal anchor is shown made up of a bracket, which is attached to the support backup studs and sheathing, and a hook.)

In addition to the two types of anchors discussed, a third alternative to secure veneer brick walls is to extend the horizontal reinforcement of the back-up walls into the veneer.

Illustrative Example – Example 2.1 in Appendix A illustrates the computation of the load from a wall on its foundation and stress in the joint mortar.

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2.3: Use of Shelf angles in Veneer Walls

Shelf angles (or relief angles) are required when veneer walls are used in multistory buildings with a total height exceeding 30 feet (9.15 meters). They are attached to the structural system, whether reinforced concrete floors, masonry bearing backup walls, wood sheathing and studs, or steel frames. Shelf angles are usually used at every other floor; and as such each angle carries the weight of veneer over two stories. Figure 2.5 illustrates an example of a shelf angle and several architectural and structural details associated with it. In particular, notice the presence of a flashing that is used to divert the moisture away from migrating inside, anchor bolts connecting the shelf angle in this special case to the concrete floor slab and spandrel beam, anchors stabilizing the veneer and preventing it from movement, and a mortar trap. In addition to proper architectural detailing, there are two important structural design considerations for a shelf angle in relation to the veneer it supports. These are (1) a proper design for spacing of the anchor bolts and their size and embedment length; and, (2) an appropriate size for the angle.

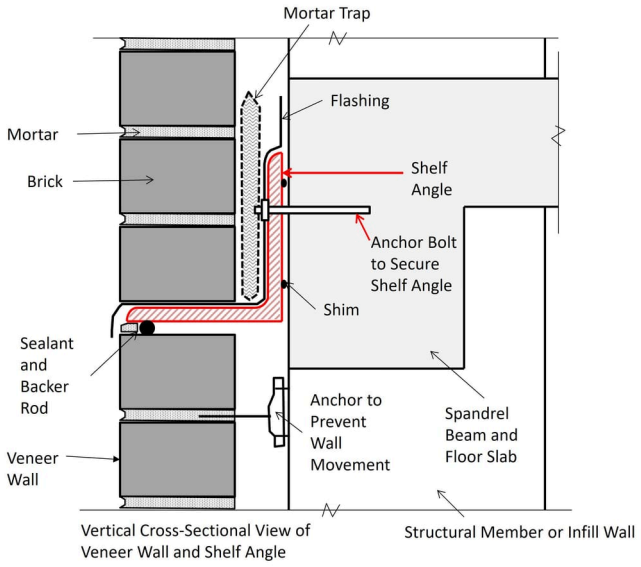


Figure 2.5 An example of a shelf angle to support veneer wall (adopted from Mohammadi, 2021)

(This figure shows the cross-section of a brick wall resting on a shelf angle. An anchor to prevent wall movement, a mortar trap, and flashing are shown. The detail of the attachment of the shelf angle to the concrete structure via anchors is also shown.)

The spacing between anchor bolts (S) and the edge distance (S_E), shown in Figure 2.6, depend on the wall configuration, its thickness, and the maximum span over which it can support itself without being damaged, see for example McGinley (2013). Furthermore, the shelf angle will need to have an adequate size to accommodate the placement of the veneer wall and be strong enough to carry the weight of the veneer without causing damage to it.

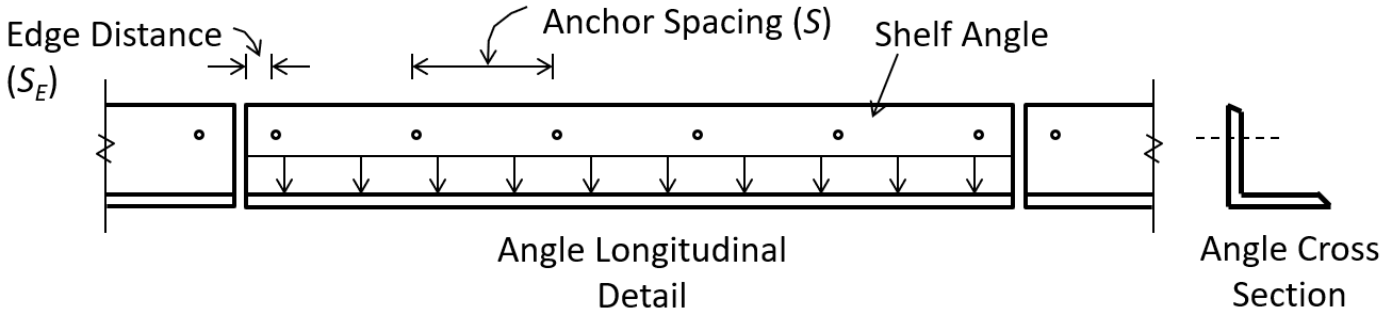


Figure 2.6 Anchor spacing and edge distance in a shelf angle

(This figure shows the longitudinal detail of a shelf angle with locations of anchors attaching it the structure and anchor spacing. The angle cross-section is shown to the right.)

The limits for the anchor bolt spacing and the edge distance can be obtained from the following equations, see Mohammadi (2021).

$$S \leq \sqrt{\frac{8T_E h F_b}{6w}} \tag{2.3.1}$$

$$S_E \leq \sqrt{\frac{T_E h F_b}{3w}} \tag{2.3.2}$$

In these equations,

- $T_E = T - 0.375$ = effective thickness of the veneer wall using inches as unit or $T_E = T - 9.5$ when using millimeters.
- T = nominal wall thickness (inches or millimeters)
- h = wall height (inches or millimeters)
- w = force from wall weight per square area, for example 42 pounds per square foot (2.0 kilonewtons per square meter) for a 4-inch (102-millimeter) wall
- F_b= Allowable tensile strength of the mortar (pounds per square inch or megapascal)

These equations are explained more in detail in Chapter 7 in discussing the structural design considerations for shelf angles. It is noted that the actual spacing and the edge distance must also consider the resistance of each anchor bolt and the applied loads. Thus Equations 2.3.1 and 2.3.2 are only used to make sure the spacing and the edge distance selected will not exceed these limits.

Illustrative examples – Examples 2.2, 2.3, 2.4 and 2.5 in Appendix A demonstrate the applications of Equations 2.3.1 and 2.3.2.

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2.4: Requirements for Veneer Design in Seismic areas

The provisions for anchored veneer seismic design are provided by ACI 530 (2013). Table 2.2 provides a summary of these provisions for various seismic categories. Note that additional requirements for seismic design are pertinent to the connection of structural elements that are used as part of façades (such as shelf angles) to concrete or masonry walls. These requirements are explained in Chapter 7 of this book.

Table 2.2 Requirements for anchored veneer design in seismic areas, ACI 530 (2013)

Seismic Design Category	Design Provision	Pertinent ACI 530 Section
C	Isolate the sides and top of anchored veneer from the structure so that vertical and lateral seismic forces resisted by the structure are not imparted to the veneer	12.2.2.10.1
D	(1) The requirements for Seismic Design Category C and the following requirements apply to anchored veneers in buildings in Seismic Design Category D (2) Reduce the maximum wall area supported by each anchor to 75 percent of that required and specified in Sections 12.2.2.5.6.1 and 12.2.2.5.6.2 of ACI 530; the maximum horizontal and vertical spacings are unchanged. (3) For masonry veneer anchored to wood backing, attach each veneer anchor to wood studs or wood framing with a corrosion-resistant $8d$ ring-shank nail, a No. 10 corrosion-resistant screw with a minimum nominal shank diameter of 0.190 inches (4.8 millimeters) or with a fastener having equivalent or greater pullout strength	12.2.2.10.2
E & F	(1) The requirements for Seismic Design Category D and the following requirements apply to anchored veneers in buildings in Seismic Design Categories E and F (2) Support the weight of anchored veneer for each story independent of other stories	12.2.2.10.3

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2.5: Design Requirements for Anchored Veneers in Areas Subject to High Wind

In areas subject to the wind velocity pressure exceeding 40 pounds per square foot (1.92 kilopascals) but less than 55 pounds per square foot (2.63 kilopascals) and the building's mean roof height of less than or equal to 60 feet (18.3 meters), ACI 530 (2013) outlines the following requirements.

- Reduce the maximum wall area supported by each anchor to 70 percent of that required in ACI 530 Sections 12.2.2.5.6.1 and 12.2.2.5.6.2.
- Space anchors at spacing not to exceed maximum 18 inches (457 millimeters) horizontally and vertically.
- Provide additional anchors around openings larger than 16 in (406 millimeters) in either direction. Space anchors around perimeter of opening at a maximum of 24 inches (610 millimeters) on center. Place anchors within 12 inches (305 millimeters) of openings.

In certain situations, when the wall segments are subject to large forces arising from the wind, horizontal structural members called girts may need to be used. These elements work in flexure carrying wind load and providing adequate distribution of the wind load to the main load bearing structural systems. Chapter 7 covers the design requirements for girts.

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2.6: Flashing, Weep Holes and Expansion Joints

Proper detailing of veneer walls requires installation of flashing, weep holes and expansion joints. As seen in veneer wall details introduced earlier in this chapter (for example in Figures 2.5), and also in details of shelf angles and lintels (see Chapter 7), flashing is always present to divert the built-up moisture or water out. The absence, or improper installation, of flashing will result in water damage to the veneer, and to the backup and interior stud walls. Flashing is a sheet or film made up of such materials as stainless steel, copper, or plastic (PVC or neoprene). In some applications, rubber films have also been used. However, it is noted that rubber films are not recommended to be used as flashing, because they deteriorate quickly.

Weep holes are provided immediately above flashing to facilitate removal of moisture that may build up behind the veneer wall. Different methods are used for constructing weep holes as described below.

Open vertical mortar joint – This is also called the open head joint and is constructed by leaving a vertical joint without mortar (see Figure 2.7). This is considered to be the most effective method to evaporate water from the cavity behind the brick wall. The spacing between open head joints is about 24 inches (610 millimeters). A screen is recommended to be used to prevent debris built-up and insect (or even small rodents) to enter the hole. Care must be exercised to properly mark the openings at the time of repair as often workers patch up the openings without realizing the main purpose for them.

Using wicks (consisting of cotton ropes) embedded in head joints – Wicks are ropes intended to absorb water from the air space by capillary action and drain it out. They are spaced 16 to 24 inches (0.4 to 0.6 meters) apart (see Figure 2.7). Commercially available wick ropes, often supplied with accompanying plastic tubes to direct moisture outward, are commonly used in masonry wall repair and reconstruction to facilitate proper drainage.

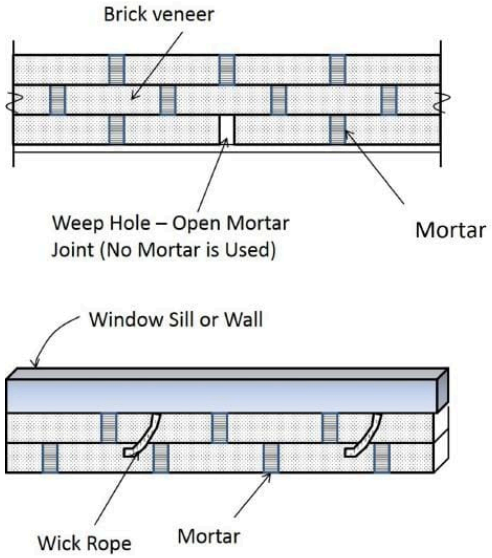


Figure 2.7 Open mortar joint, weep holes and wicks

(This figure shows two details. The one on the top shows the facing view of a segment of a brick wall with a mortar joint left unfilled for moisture escape. The one on the bottom, shows the facing view of a segment of a brick wall below a window sill with two wick ropes extending out for moisture escape.)

Vertical expansion joints – Vertical expansion joints are used to avoid cracking due to expansion and contraction. Generally, they are constructed about 10 feet (3.05 meters) from the corners and about 25 feet (7.62 meters) apart from each other (see Figure 2.8 for a horizontal cross section of a wall showing the vertical expansion joint). The lack of expansion joints will result in crack development in the wall. Cracks develop when the wall goes through expansion and contraction because of temperature variation and foundation movement and settlement. The computation of the width of an expansion joint can be done based on the variation in the temperature, from a lower extreme to an upper extreme (ΔT), and the coefficient of thermal expansion of the wall material. Conversely, a value for the width of the joint may be selected and using the temperate change (ΔT), the distance

between adjacent expansion joints can be computed.

Considering the coefficient of thermal expansion as α, which has a unit one over degree Farenheit (or one over degree Celsius), and a value for the width of the joint as b_j the elongation over a length L for a segment of a wall between two joints will be

$$\Delta L = \alpha \times L \times (\Delta T) \tag{2.6.1}$$

For a wall segment in between two adjacent joints, thus taking $\Delta L = b_j$, the limit for the distance L will be

$$L \leq b_j/(\alpha \times \Delta T) \tag{2.6.2}$$

The limit for L for the first joint from the corner of the wall is one half of the value in Equation 2.6.2, since the wall segment is considered fixed at one end and is allowed to elongate only one half of the value decided for b_j is used for this situation.

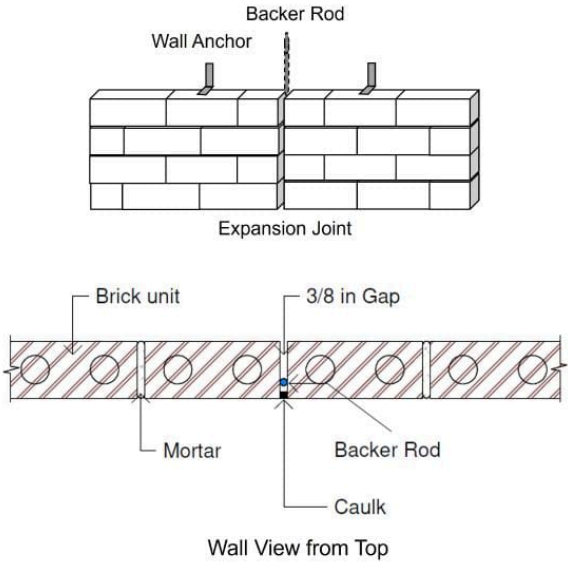


Fig. 2.8 Details of a vertical expansion joint

(This figure shows a facing view of a brick wall with a vertical expansion joint and a backer rod situated within the joint. A sketch below this view shows the wall cross-section, viewed from the top, with details of an expansion joint including the backer rod and caulk.)

Table 2.3 provides the values of the coefficient of thermal expansion for several wall materials,

Table 2.3 Coefficient of Thermal Expansion (α)

Material	Per 1 degree Farenheit	Per 1 degree Celsius
Brick Masonry	3.1 times 1o to the power of -6	5.6 times 1o to the power of -6
Mortar	4.1 to 7.5 times 1o to the power of -6	7.3 to 13.5 times 1o to the power of -6
Concrete and CMU	7.5 times 1o to the power of -6	13.5 times 1o to the power of -6
Reinforced Concrete	5.4 times 1o to the power of -6	9.8 times 1o to the power of -6

Wood	Parallel to grain: 1.7 times 1o to the power of -6 Perpendicular to grain: 16.7 times 1o to the power of -6	Parallel to grain: 3.0 times 1o to the power of -6 Perpendicular to grain: 30.0 times 1o to the power of -6
Glass Pane	5 times 1o to the power of -6	9.0 times 1o to the power of -6
Steel	6.5 times 1o to the power of -6	11.7 times 1o to the power of -6

It is noted that expansion joints are essential for all types of walls, whether they are veneers or load-bearing structures. When construction fails to include these joints, the wall will naturally develop cracks, typically spaced at regular intervals. These cracks are usually vertical and act as unintended expansion joints, as the wall essentially forces itself to relieve pressure by cracking.

Illustrative example – Example 2.6 in Appendix A demonstrates the application of Equations 2.6.1 and 2.6.2.

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2.7: Damage to Brick Walls and Repair Methods

A variety of reasons may contribute to veneer wall damage. Usually, damage detection is only possible conducting a thorough visual inspection from close distance atop scaffolds. In this section, a general overview of damage scenarios that are more common is presented. However, it is emphasized that any one case may be unique and different from others. Thus, it is imperative that a detailed plan for inspection is prepared. Such a plan will need to include the procedure for close-up inspections, prescription of appropriate non-destructive test and evaluation methods that may be necessary to ascertain the condition of critical areas, follow-up actions in a short and long-term basis, and suggesting repair methods that are cost effective and appropriate to the type of damage identified.

It is noted that in addition to bricks that make up a major portion of a veneer wall, there may be other types of pieces and elements that experience damage. And as such, it is imperative that an inspection include the entire veneer and all of these additional pieces and components that may also be present. Examples are ornamental stone pieces, wood used as frames around openings, terra-cotta pieces, iron pieces used as flagpoles, commercial signs, external fire escapes, porches and balconies, and coping stones. Because of their importance to the overall safety of a building envelope system, some of these additional components are covered elsewhere in this book.

Brick and mortar damage – This can be in the form of

- Missing and deteriorated bricks.
- Cracks that extend through mortar.
- Mortar lost or washed out.
- Efflorescence on bricks.
- Dislocation of bricks in a segment of the wall.

Missing and deteriorated bricks are generally because of weather conditions, and ice and freeze/thaw cycles. These types of damage are mainly detected by close examination of the façade atop a scaffold or by afar observation using binoculars. The repair method often requires replacing damaged bricks and applying new tuckpointing.

Cracks in a veneer wall can also be because of the weather conditions and temperature effects causing expansion and contractions. Cracks also can be because of excessive deflection of lintels and shelf angles and foundation settlement. In some situations, vibrations can also cause cracks. This may especially be prevalent when there is a prolonged vibration induced on the wall because of construction activities, trains operating nearby or even frequent airplane landing and taking off activities in the vicinity of the building. The best method for crack detection is by close examination atop a scaffold. However, to ascertain the cause of cracks, more detailed examination using measurement devices (for an extended period of time) and non-destructive test and evaluation methods may be necessary, see Chapter 8.

The repair method may involve a combination of actions to not only bring the façade to the original shape but also to prevent the damage from happening again. Thus, a detailed analysis of the situation may be necessary to ascertain the most probable cause of cracks. For example, if the foundation settlement is determined to be the cause of cracks, underpinning may be necessary in addition to thr repair and replacement of a segment or the entire wall. Underpinning is a major effort in stabilizing foundations and preventing the recurrence of any additional settlement over time and involves installing elements called micro-piles beneath the foundations.

Figure 2.9 illustrates several kinds of cracks as observed in brick walls. Figure 2.9(a) shows a crack in a corner most likely generated because of expansions and contractions. Figure 2.9(b) shows a diagonal crack above a lintel. The most likely cause of this crack can be attributed to the excessive deflection of the lintel especially when the lintel is subject to vibration causing fluctuation of stress in the masonry (see for example, Longinow and Mohammadi, 2022). Usually, these cracks are two-directional forming an approximate isosceles triangle of 45 degrees; however, at times, and especially when the lintel is at a corner, such as the situation depicted in Figure 2.9(b), the crack can be one-directional. Figure 2.9(c) shows a vertical crack in the mortar caused by some kind of movement (such as temperature variations); while the cracks shown in the brick are most likely caused by weathering as well as expansions and contractions. Figure 2.9(d) shows cracks around a metal piece left in the wall during remodeling. Corroded metals expand causing cracks in the surrounding façade masonry elements. Figure 2.9(e) shows a vertical crack below the right support in a lintel. The most likely cause of this crack is attributed to a heavy reaction force from the lintel or expansions and contractions.

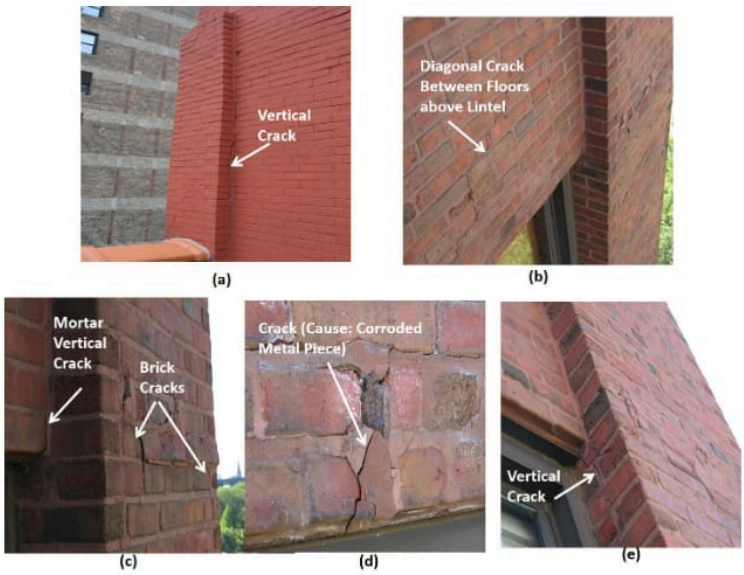


Figure 2.9 Brick wall crack examples (photos courtesy of Dearborn Engineering Corporation)

(This image shows five figures – two on the top and three on the bottom. The top figure on the left shows a vertical crack at a corner of a brick wall. The one on the top right shows a diagonal crack in a brick wall above a window. The lower figures show various cracks in brick walls).

Mortar loss or washouts are generally a result of weather conditions, water runoff on walls and snow and ice built-ups. Low quality mortars can also be another cause. These conditions are generally detectable by afar examination or in a more detailed way by close examination of affected areas. Figure 2.10 illustrates a mortar washout in a brick veneer wall in a highrise building.



Fig. 2.10 Illustration showing mortar washout (photo courtesy of Dearborn Engineering Corporation)

(This figure shows a photograph of mortar washout in a brick wall.)

Efflorescence (sometimes referred to as “whiskers”) is a white powdery patchy deposit appearing on brick walls (see Figure 2.11). Generally, three conditions must exist for the efflorescence deposit to form. First, there must be water-soluble salts present somewhere in the wall; second, there must be sufficient moisture in the wall to render the salts into a soluble solution; and third, there must be a path for the soluble salts to migrate through to the surface where the moisture can evaporate, thus depositing the salts, which then crystallize and cause efflorescence (see Figure 2.11). Efflorescence deposits are generally observed easily by

afar inspection.

The efflorescence repair first starts by eliminating one or more of the three conditions (usually the moisture) and then is followed by washing the affected area using an appropriate chemical compound. The washing is often done by water under pressure or by using one of several compounds recommended for removing efflorescence. The use of muriatic acid is also recommended following by flushing the affected area with water. Before applying acid to brick masonry, wetting of the area will need to be done to avoid "burning" or discoloration of the brick or mortar damage. Some light sandblasting for removal of stubborn efflorescence has also been recommended. However, this needs to be done carefully for brick to avoid creating porous surfaces; and the method may be more appropriate for concrete surfaces.

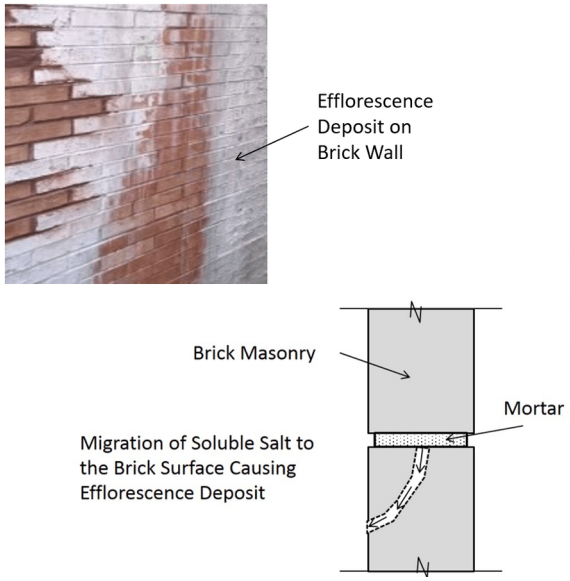


Figure 2.11 Efflorescence deposit on a wall and its formation process (photo courtesy of Dearborn Engineering Corporation)

(This figure shows a photo of efflorescence deposit on a brick wall on the left. On lower right, it shows a cross-section of a portion of a wall in vertical direction demonstrating the path for the migration of soluble salt to exterior of a wall.)

Dislocation of brick veneer walls appears in the form of a bulge. It is often detected via close examination from a corner of the building. A major cause of the veneer bulging is improper installation and lack of, or damage in, support anchors that are needed to secure the veneer wall to the backup system (masonry wall, structural system or sheathing). As seen in Figure 2.12, for example, the improper installation of the anchor has resulted in anchor losing its contact with the masonry and becoming ineffective in maintaining the stability of the brick wall. The repair will require reconstructing the wall with proper anchors installed as described earlier in this course.

Figure 2.13 illustrates a case of dislocated brick wall. While with afar examination the wall bulging out may not be evident, upon further examination from the roof (as shown in the photo on the right in Figure 2.13), the bulging is clearly observed. This damage scenario is most likely resulted from the lack of, or deteriorated, support anchors to prevent the wall movement. Furthermore, signs of damage to the mortar and bricks are also visible on the left in Figure 2.13.

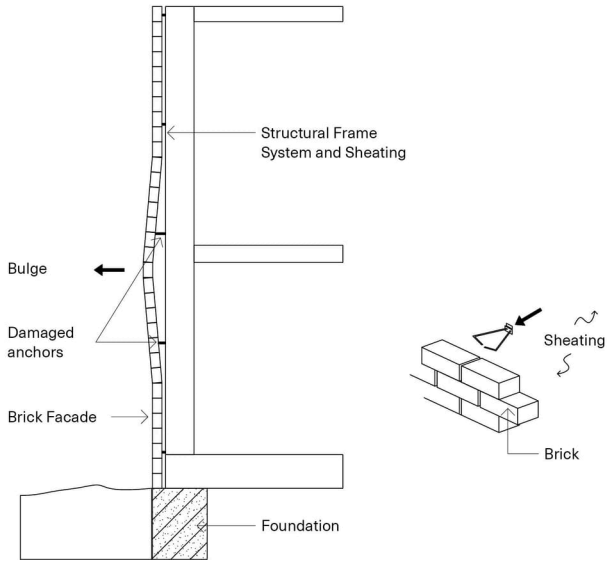


Figure 2.12 Damage to brick veneer wall (lack, or improper installation, of anchors)

(This figure shows the vertical detail of a brick veneer wall bulging out. To the right, a detail of a segment of a brick wall is shown with an anchor that has no contact with the wall causing the bulging.)

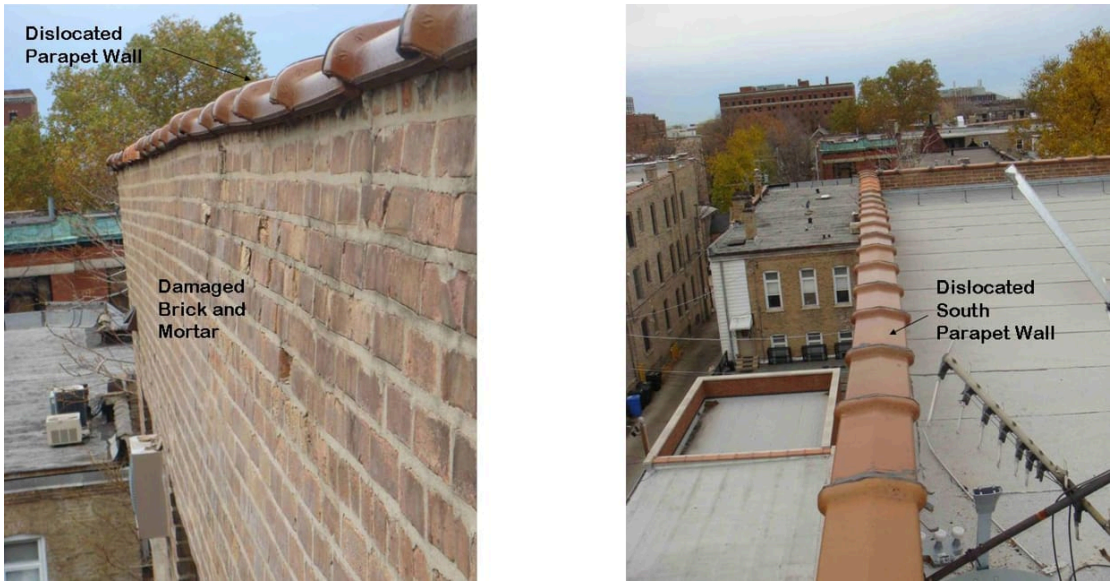


Figure 2.13 Damage to a brick wall in the form of bulge (photos courtesy of Dearborn Engineering Corporations)

(This image shows two photos. The one on the left shows a parapet wall dislocated. The one on the right shows the same parapet wall from roof with a clearer view of wall dislocation.)

Lintel and shelf angle damage – Lintels and shelf angles are common structural members used in façades. Lintels span over openings above windows and doors; while shelf angles provide support for veneer walls (as explained in Section 2.3 and also in Chapter 7). Damage in lintels and shelf angles is often hard to detect without creating any additional openings in the affected area and exposing these structural members. The damage is generally in the form of corrosion, which results in loss of effective cross-sectional areas. A major cause of such damage is infiltration of water through the masonry, which can be because of lack, or improper installation, of flashing. The detailing of the flashing at the ends of a lintel is especially important to create dams to prevent water infiltration through corners of an opening (such as a window). The repair requires removal of the masonry and properly grind and coat the structural member and install flashing as depicted, for example in the detail of Figure 2.5. The prevalence of corrosion in shelf angles and lintels may require a complete structural analysis and redesign of these components

and their attachment to the structural system. Chapter 7 covers the procedures for shelf angle and lintels structural design including the connection design of shelf angles to structural members.

Window-sills and window frame damage – Wood sills and frames may damage because of periodic wet/dry conditions, insect/bug infestation and lack of proper maintenance and painting, and corrosion. Repair methods may range from cleaning the affected elements and repainting to a complete replacement. Stone window sill damage often appears in the form of loss of mortar beneath, cracks or chipped pieces and can be because of improper installation of flashing, lack of backer rods and wicks, and neglect in conducting periodic inspections. Again, repair may range from simple cleaning and tuckpointing to complete replacement with proper installation of flashing, backer rod and wicks.

Figure 2.14 illustrates three cases of window damage. Figure 2.14(a) shows a corroded window frame and damage to the façade, which most probably has been the effect of metal corrosion. Figure 2.14(b) depicts an improper prior repair; while Figure 2.14(c) shows still yet another case of corrosion damage to a window frame.

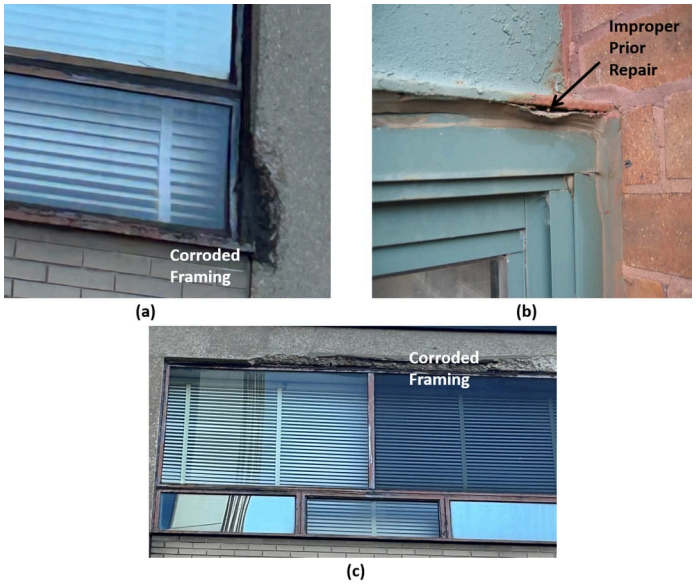


Figure 2.14 Examples of window damage

(In this figure, three photos show corrosion and improper repair causing damage around windows.)

Ornamental stone and terra-cotta damage – Damage in these elements appear in the form of cracks, crazing, and loss of the attachment rods that secure them to the structural members. Damage to rods is often due to moisture and corrosion. Terra-cotta blocks have also been used as major elements of façades especially between the late 19th century and the 1930s. These blocks offered architects with a modular, varied and relatively inexpensive approach to façade construction. Figure 2.15 shows cracks in terra-cotta ornamental blocks around a window and a case of improper repair. Because of their role in making up façades in many buildings that are in use today, terra-cotta façades are covered more in detail in Chapter 4.



Figure 2.15 Terra-cotta damage examples (photos courtesy of Dearborn Engineering Corporation)

(Two photos are shown in this figure. The one on the left shows a crack in a terra-cotta block. The one on the right shoes patchy surface cracks called crazing on a terra-cotta block.)

Table 2.4 summarizes several common damage scenarios often observed in brick masonry, probable causes of damage, and suggested methods for inspection and repair.

Table 2.4 Common types of brick façade damage

Type of Damage	Probable Causes	Method of Inspection	Repair Method
Deteriorated mortar	Washout; substandard quality; heavy load; water penetration; frequent freeze/thaw cycles	Afar examination with powerful binoculars; close examination atop scaffold	(1) Remove old damaged mortar; (2) Clean area; (3) Apply new tuckpointing
Deteriorated brick	Frequent freeze/thaw cycles; substandard brick units	Afar examination with powerful binoculars; close examination atop scaffold; may also require exploratory openings by removing blocks to examine any signs of moisture behind the wall	(1) Remove damaged blocks; (2) Clean area; (3) Replace with new blocks
Cracked or dislocated brick	Heavy load; loss of support anchors causing wall to bulge; wall or foundation settlement; wind load effect; improper lintel design; corroded embedded metal	Close examination atop scaffold; may also require exploratory openings by removing blocks to examine any sign of support anchor damage and lintel condition	(1) Remove damaged blocks or wall segments; (2) Repair damaged or missing support anchors; (3) Rebuild wall Note: long vertical cracks in a consistent pattern may be due to lack of expansion joints, which requires rebuilding with proper expansion joints

Damaged shelf angle or lintel (also, see Chapter 7)	Corrosion because of water penetration or leak; inadequate design; heavy load; improper or lack of flashing and weep holes; improper prior repairs	Close examination atop scaffold; generally, requires exploratory openings by removing brick blocks to expose shelf angle and lintel completely to determine the extent of damage	(1) Grind and sand to clean rust, if damage minor and the element structurally sound; (2) Replace if damage extensive or structurally deficient (temporary shoring required for proper handling the repair)
Efflorescence deposit	Existence of water-soluble salt in wall along with moisture buildup; substandard bricks have a higher incidence of efflorescence	Afar examination	(1) Remove source of moisture (generally this can be done by installing weep holes and wicks); (2) Replace bricks if substandard; otherwise wash area using an appropriate chemical compound (application of muriatic acid is recommended following by flushing with water)
Ornamental stone and terra-cotta damage (also see Chapter 4)	Cracks and loss of the attachment rods that secure them to the structural members; grazing; spalling	Afar examination with powerful binoculars; close examination atop scaffold; may also require exploratory openings by removing blocks to examine any sign of moisture behind the wall and corrosion in the attachment rod	Minor damage: clean the area and apply coating and/or fill cracks with quality sealant materials Major damage: (1) remove spotty (1) Remove damaged blocks or ornamental pieces; (2) Repair damaged or missing support rods; (3) Remove source of moisture; (4) Replace and/or rebuild wall
Window and window-sill damage	Water wash-out; improper detail; lack of weep holes; wicks and backer rod; corroded window frame	Close visual inspection atop scaffold	Repair may include simply cleaning the area and caulking or complete replacement correcting the detail; frame replacement may be necessary in severe corrosion cases
Parapet wall damage	Damaged coping stone; lack of proper flashing; inadequate detail on the roof side (roof membrane termination detail)	Close visual inspection from roof	Repair may simply involve correcting the detail, adding flashing and new coping stone; or complete replacement when damage is major

Issues related to ancillary structures (balconies, porches, fire escapes, flags and signs)	In addition to deterioration and damage that are unique to ancillary structures (such as wear and tear, corrosion or structural deficiencies); the points of attachment of these structures to the wall may have damage in the form of loose connections, missing or cracked bricks; loss of mortar and missing or damaged fasteners and anchors	Close examination atop scaffold or (in the case of a fire escape or porch) climbing down the steps inspecting all connections and attachments closely	Major damage requires (1) Cleaning the area; (2) Replacing missing or damaged anchors; connections, fasteners, etc., and (3) Applying new mortar and new coating and paint

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2.9: Exercises

E2.1 While inspecting a façade, we decide that the condition of a shelf angle is not satisfactory; and it will need to be replaced. The façade is made up of bricks. The shelf angle in question carries the weight of a wall that is only 6 feet high. The decision is to use an angle 5-inch by 3.5-inch with a quarter of an inch in thickness (127-millimeter by 89-millimeter and 6 millimeters in thickness). The 5-inch leg is outstanding, while the 3.5-inch leg is vertical. The wall is a veneer of 4 inches in thickness. A spacing equal to 6 feet (1.8 meters) with 2 feet (0.6 meters) of edge distance for the anchors are used. For the mortar resistance, F_b=15 pounds per square inch (103 kilopascals) is conservatively suggested.

a) Based on the information given, determine the limit for the spacing S and decide whether the 6 feet spacing is acceptable.

b) Determine the limit for the edge distance S_E and decide whether the 2-foot edge distance is acceptable.

c) Determine the minimum thickness needed for the angle. You can neglect the angle's self-weight. Does the angle selected have an accepted thickness? The cavity between the veneer and the back-up support system is 1.5 inches (38 millimeters).

E2.2 The 2-piece anchors for attaching veneer walls to backup walls prevent differential movement in

a) All directions.

b) Vertical direction only.

c) Horizontal and vertical directions.

d) A direction perpendicular to the plane of the veneer wall.

E2.3 A diagonal crack appears in the north-façade brick wall running over an entire height of the first floor of a three-story building. This crack is most likely because of

a) Rainwater running over the wall.

b) Freeze/thaw cycles.

c) Wind pressure perpendicular to the plane of the wall.

d) Partial foundation settlement.

E2.4 On the south façade of the building in E2.3, the brick veneer wall is bulged out at the middle of the wall. The displacement runs from roof to the ground level. The most likely cause of this displacement is

a) Excessive wind load.

b) Excessive settlement of the footing.

c) Inadequate anchors attaching the veneer to the backup wall.

d) Low grade mortar.

E2.5 Flashings are used

a) At the foundation level.

b) Over shelf angles.

c) Over lintel angles and under window sills.

d) At all of the locations indicated in (a), (b) and (c).

E2.6 Which of the following two shelf angle designs would be most appropriate in a building?

a) A continuous shelf angle running over 40 feet (12.2 meters) made up of welded pieces.

b) A shelf angle made up of 20-foot pieces (6.1 meters) with 3-8th of an inch of gap between adjacent lengths.

c) None.

d) Both.

E2.7 Which of the following statements is *incorrect*?

a) Type *N* mortar has about 750 pounds per square inch (5.2 megapascals) compressive capacity.

b) A new brick veneer wall and footing (made up of normal weight concrete) have about the same compressive capacity.

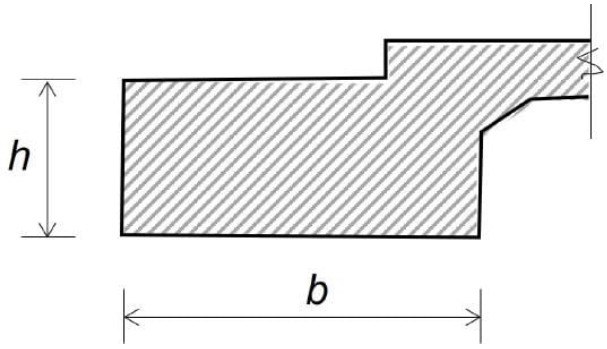
c) A backer rod is used in joints where slight elastic movement is needed.

d) Weep holes can be eliminated in veneer walls in low-rise buildings.

E2.8 The accompanying figure shows the cross section of a wall foundation where h=10 inches (254 millimeters) and b= 14 inches (356 millimeters). The allowable soil bearing capacity is 2,500 pounds per square foot (119.7 kilonewtons per square meter). The foundation carries the weight of a 4-inch veneer that is 10 feet (3.05 meters) high and the weight of an 8-in back-up CMU wall that is also 10 feet (3.05 meters) high.

a) Determine the total load W_T applied from the foundation to the soil below using the unit of pounds per foot (or kilonewtons per meter).

b) Compute the pressure applied to the soil in pounds per square foot (or kilopascal) and decide whether the size b selected is ok.



(This figure shows a cross-section of a foundation for a wall with a width b and a height h.)

E2.9 Which of the following statements is *incorrect*?

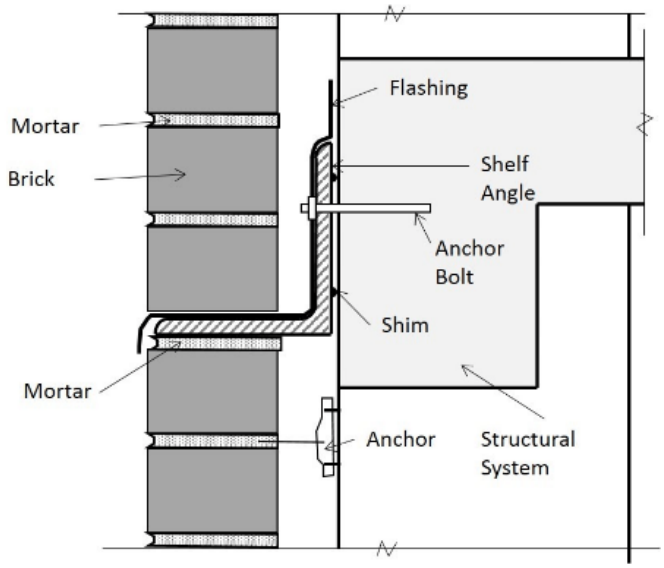
a) The safety in the LRFD method is based on applying the load and resistance factors.

b) The ASD method uses load factors but does not include resistance reduction factor.

c) The basis of the ASD method is in using an allowable value for the resistance.

d) When we compute deflection of a shelf angle, we never use load factors.

E2.10 Examine the following detail that shows the vertical cross-section of a brick veneer wall.



(This figure shows the cross-section of a brick wall resting on a shelf angle. An anchor to prevent wall movement, a mortar trap, and flashing are shown. The detail of the attachment of the shelf angle to the concrete structure via anchors is also shown.)

In your opinion, what may the deficiency of this detail be?

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3: CONCRETE MASONRY UNITS (CMU) AND LOAD-BEARING WALLS

Chapter Synopsis

This chapter covers inspection of walls made up of Concrete Masonry Units (CMU) blocks, all-concrete walls, and other types of load-bearing walls including those made up of tilt-up panels. Potential damage areas of these kinds of walls, and their repair methods are also covered.

3.1: Introductory Remarks

3.2: Reinforcement for Brick and CMU Walls

3.3: Bond and Ledger Beams

3.4: Design Requirements for Reinforced Concrete Walls

3.4.1: Simplified Wall Design

3.4.2: Wall Shear Resistance

3.5: Bearing Wall Distress Conditions and Repair Methods

3.5.1: Distress in Masonry Walls

3.5.2: Distress in Concrete Walls

3.6: Tilt-up Panels used as Façades

3.6.1: Distress Conditions in Tilt-up Panels

3.7: Chapter References

3.8: Supplementary Materials to this Chapter

3.9: Exercises

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3.1: Introductory Remarks

Bearing walls provide support to roofs and floors. They are generally used as backup walls; however, in some buildings they may also appear as façades. In most modern applications, these walls are reinforced with two types of reinforcement: (1) longitudinal reinforcement, which is made up of rebars along the height of the wall; and (2) transverse reinforcement, which is a grid-type placed along the length of the wall. In this chapter, bearing walls made up of, concrete masonry units (CMUs), all-concrete and other materials (such as load-bearing bricks and walls made up of tilt-up panels) are presented.

Tilt-up panels are cast horizontally and then lifted into place to form the building façade and complete the structure. Sufficient reinforcement is incorporated to meet strength requirements during both the construction and lifting phases and to support applied loads after the panels are erected. In situations where short concrete brackets are built integrally with the wall for the roof and floor beam support, special reinforcement, with proper detailing, is needed for the brackets. Additional reinforcement may also be needed around openings and near the metal brackets that connect adjacent panels. These locations are stress-concentration points and have potentials for crack development. As with other concrete walls, the main reinforcement consists of a grid of vertical and horizontal rebars.

This chapter primarily covers walls that also work as façades and focuses on topics that are relevant to façade inspections, identification of the types and sources of damage, and common repair methods that can be prescribed as part of the building envelope rehabilitation.

Loads on bearing walls – The loads on bearing walls generally consist of

- **Dead, live and snow loads transferred from the roof and floors**: These loads will usually result in an axial force applied on the cross section of the wall.
- **In-plane lateral load (in-plane shear force) from wind and seismic effects**: This load is often prevalent when the wall is also acting as a shear wall.
- **Out-of-plane lateral load (out-of-plane shear force)**: This is present when a lateral force acts perpendicular to the wall plane or when the axial compression force on the cross section is eccentric. When axial compression load is eccentric, an out-of-plane moment is developed. This out-of-plane moment results in an out-of-plane shear as well. Another example of an out-of-plane shear force in a wall is when a lateral force acts perpendicular to the wall plane in situations when the wall is subject to soil pressure (such as, for example, in a basement where a wall is partially below the ground).
- **In-plane moment**: This is resulting from an in-plane eccentricity of the axial force or from an in-plane shear force.
- **Out-of-plane moment**: This is resulting from an-out-of-plane eccentricity of the axial force or from an out-of-plane shear force.

Figure 3.1 shows all potential forces that can be present in a load-bearing wall.

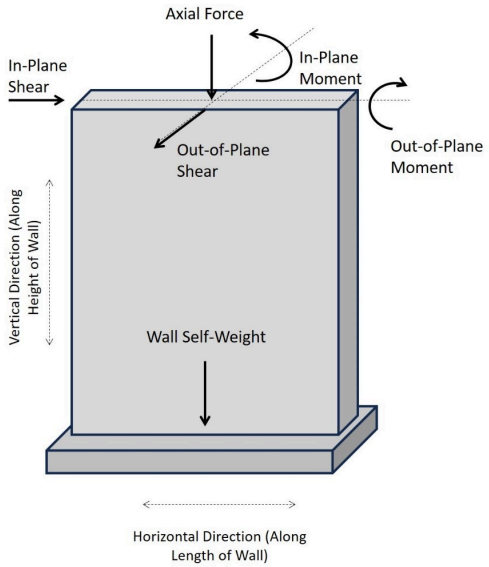


Figure 3.1 Potential forces applied on a bearing wall

(This figure shows a wall segment with all potential forces applied on it. Forces shown with arrows are a vertical force on the top, an in-plane moment, an out-of-plane moment, an in-plane shear, an out-of-plane shear and a load from the wall self-weight on the bottom).

When a plan for repair of a load-bearing wall is developed, it is imperative that the aforementioned loads are properly considered and accounted for. When using the LRFD method, we need to combine various loads with appropriate load factors to determine the most critical loading conditions. Various load factors and load combinations and the resistance reduction factors for concrete applications are presented in the supplementary section to this chapter. Similar load and resistance factors are also used in masonry systems.

Figure 3.2 shows the detail of a masonry wall supporting a floor. Under the application of the floor and roof loads, which are called gravity loads, the masonry wall will need to be designed using the provisions of the ACI 530 (2013). Generally, this is the case when the gravity load is heavy and there are potentials for lateral loads. Otherwise, a minimum amount of reinforcement recommended for masonry walls will be adequate.

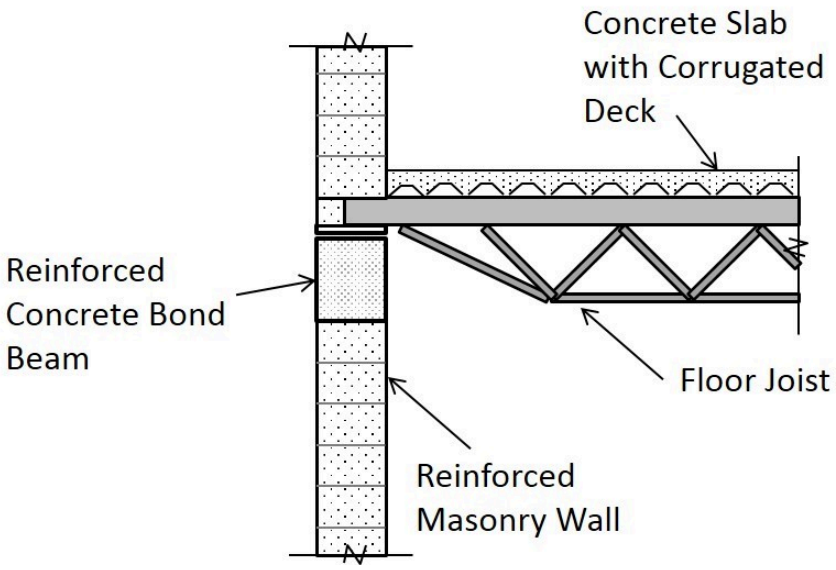


Figure 3.2 Masonry wall supporting a floor

(This figure shows a vertical cross-section of a wall with floor detail. Floor details show a floor joist in the form of a truss and a concrete deck. The wall detail shows a concrete bond beam supporting the floor).

As shown in Figure 3.2, the floor system is made up of steel joists that are supported by a reinforced concrete bond beam. Bond beams can also be in the form of special-shaped masonry blocks filled with grout and reinforced with longitudinal bars (as explained later). Often the joists are secured to the bond beam with anchors to prevent their movement. Other alternate floor system options may be

- One that is made up of timber sections attached to the wall through elements called joist hangers.
- Steel profiles such as channels and W sections along with a reinforced concrete floor slab.

A bearing wall rests on a continuous foundation. The foundation needs to be designed properly for its size and reinforcement. Key factors in sizing the foundation are the soil bearing capacity, and the pressure that is applied on the supporting soil by the total load transferred from the wall, including the self-weights of the wall and the foundation. If the foundation is buried at some depth from the ground surface, which is generally the case especially in cold regions, the weight of the soil (surcharge) must also be considered in the design for the size of the foundation. When calculations are performed for sizing the foundation, no load factors are generally applied; and the allowable soil bearing capacity is used in the computation. However, the reinforcement design is based on factored applied loads and the resistances of the bars and concrete following the provision of the ACI code for the "strength method."

Some concrete bearing walls may also be subject to a lateral force (perpendicular to the plane) because of the soil pressure. This will be, for example, in a building that is on a slope; and as such part of the façade may be buried under the ground. In situations like this, the stability of the wall and its foundation will also need to be investigated, if any kind of damage, such as cracks, is attributed to the soil pressure.

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3.2: Reinforcement for Brick and CMU Walls

As indicated before, brick masonry and CMU bearing walls will need to be properly designed for the applied loads, especially when lateral loads are present, following applicable design codes. This section only focuses on the type, and appropriate detailing, of reinforcement that is generally needed when a wall goes through repair and/or rehabilitation. The reinforcement is made up of bars running in both longitudinal (along the height of the wall) and transverse (along the length of the wall) directions. Please note that with these definitions, the longitudinal reinforcement is actually vertical and transverse reinforcement is horizontal. Furthermore, the masonry blocks are filled with concrete-like grout in about every fourth core where the longitudinal rebars are located; and as such, the rebars are practically embedded in grout thus maintaining the bond between the steel and the masonry materials. The transverse reinforcement is usually in the form of a latticed bar and is placed in the mortar joints at every other course. Figure 3.3 illustrates the schematics of the longitudinal and transverse reinforcement.

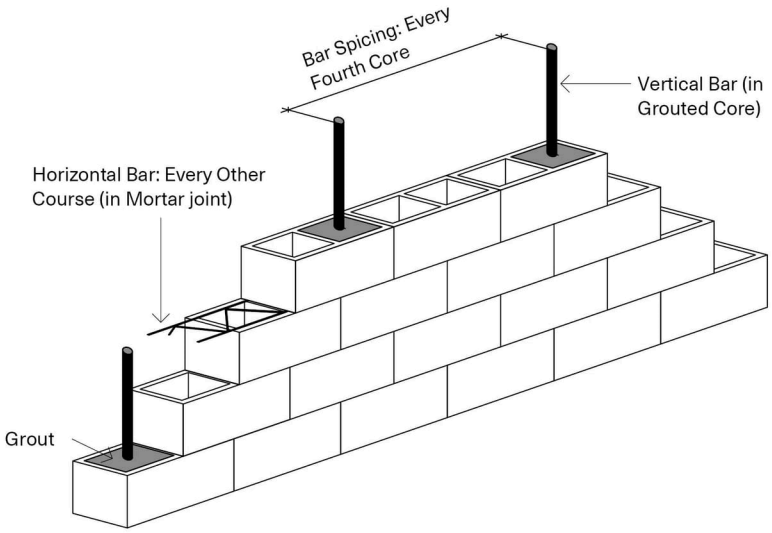


Figure 3.3 Longitudinal (vertical) and transverse (horizontal) reinforcement in masonry walls

(This figure shows a segment of a masonry wall made up of hollow masonry blocks. Vertical and horizontal reinforcements are shown. Horizontal reinforcement is in the form of latticed system.)

The rebar sizes are generally as described below (if no unusual loads such as wind or seismic are considered critical).

Longitudinal reinforcement (vertical bars) – These are usually #4 or #5 bars at every fourth core filled with concrete-like grout. Note: each #4 bar has 0.5 inches (13 millimeters) of diameter and 0.2 square inches (129 square millimeter) of cross-sectional area. The diameter and cross-sectional areas for #5 bars are 5-8th of an inch and 0.31 square inches (16 millimeters and 200 square millimeters), respectively.

Transverse reinforcement (horizontal bars) – These are often in the form of prefabricated lattice bars using gage 9 or 10, see Table 3.1 for the gage information using the American Wire Gage (AWG) designation.

Table 3.1 Wire gages using AWG designation (Note: not all available gages are listed)

Gage	Diameter, inch (millimeter)
1	0.2893 (7.3)

2	0.2575 ≈ 1/4 (6.5)
6	0.1620 (4.1)
7	0.1443 (3.7)
8	0.1285 ≈ 1/8 (3.3)
9	0.1144 (2.9)
10	0.1019 (2.6)
15	0.0571 (1.5)

There are two types of CMUs, Type N and S. Type N is used for above or below the ground; while Type S is for above the ground applications only. Each CMU is about 16 inches (406 millimeters) long and about 8 inches (203 millimeters) high. The width can be 4, 6, 8, 10 or 12 inches (102, 152, 203, 254 or 305 millimeters). For 2 to 3 story buildings in normal applications, the width is usually 8 inches (203 millimeters).

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3.3: Bond and Ledger Beams

As briefly explained earlier, in situations such as the one in Figure 3.2, when the floor or roof joists are positioned directly over CMU or brick blocks, a bond beam is used to accommodate a proper support system for the joists. Bond beams are generally made up of either reinforced concrete or U-shape reinforced CMU blocks. Reinforced concrete bond beams are positioned with a width equal to the thickness of the wall or smaller. They are generally reinforced with a minimum amount of longitudinal steel and transverse ties. In the example of Figure 3.4, 4#5 bars and #3 ties are used. If the bond beam bridges over an opening, we must make sure the reinforcement is adequate for the bending moment developed in the beam since it will be under flexure and shear. Occasionally, bond beams, especially on the roof, may also be subject to a torsional moment. In that case, we must make sure the beam is stable and is properly reinforced to prevent cracks developed as a result of twisting.

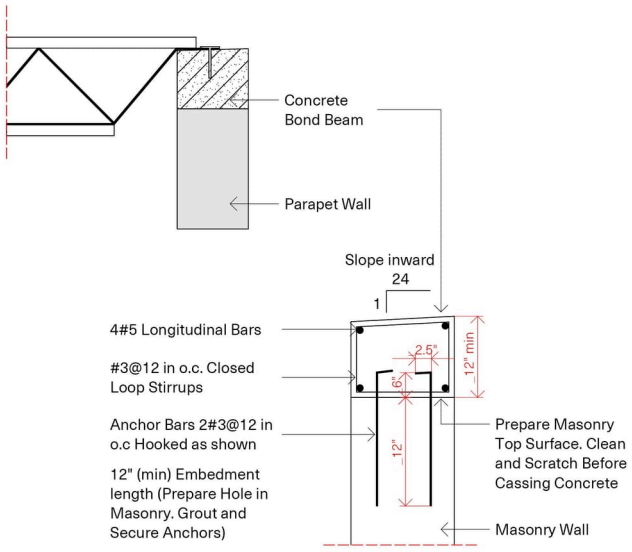


Figure 3.4 An example of a reinforced concrete bond beam

(This figure shows two details. On the left, a portion of a prefabricated joist is shown resting on a wall with a concrete bond beam. On the right an enlarged detail and reinforcement of the concrete bond beam is shown.)

An examples of a reinforced masonry bond beam using U-shape CMU blocks is shown in Figure 3.5.

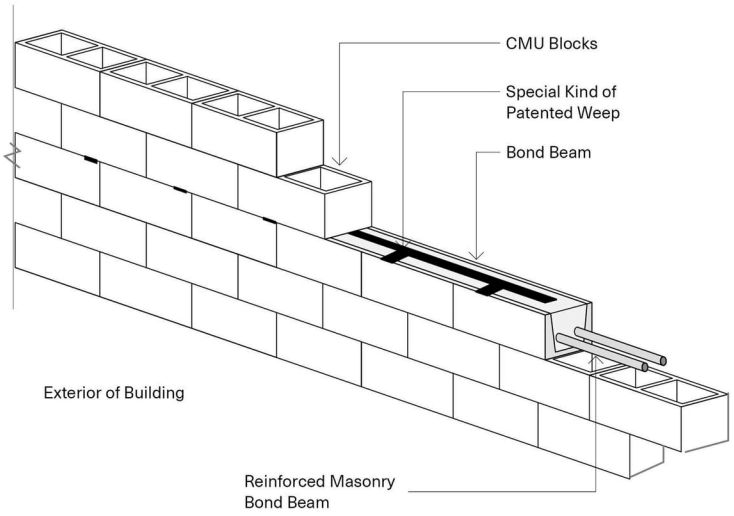


Figure 3.5 Masonry wall with reinforced bond beam

(This figure shows a three-dimensional view of a segment of a reinforced masonry wall with a reinforced masonry bond beam.)

Figure 3.6 shows two types of blocks that are often used in bond beams.

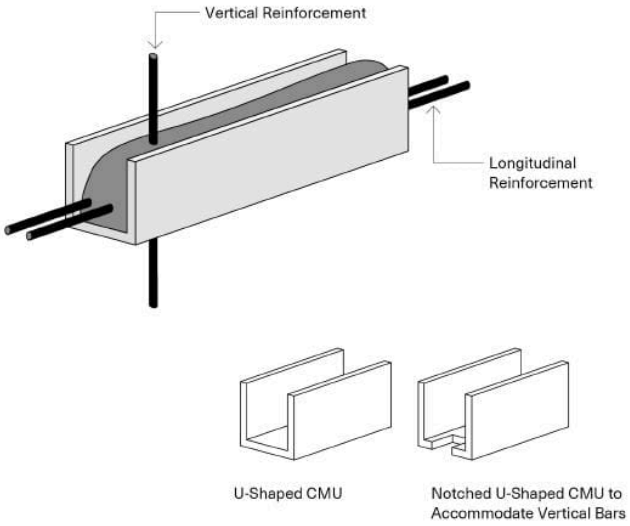


Figure 3.6 CMU blocks used to make bond beams

(This figure shows details of reinforcement un a U-shaped masonry bond beam. Images of two kinds of U-shaped blocks used to form a bond beam are shown to the lower right.)

In certain cases, floor and roof joists will need to be attached to the wall using joist hangers. In those situations, a timber beam, usually with a nominal thickness of 2 inches (51 millimeters) is secured to the wall via anchor bolts. This beam is referred to as the ledger beam. Hangers are then attached to it and provide support for the joists (see Figure 3.7).

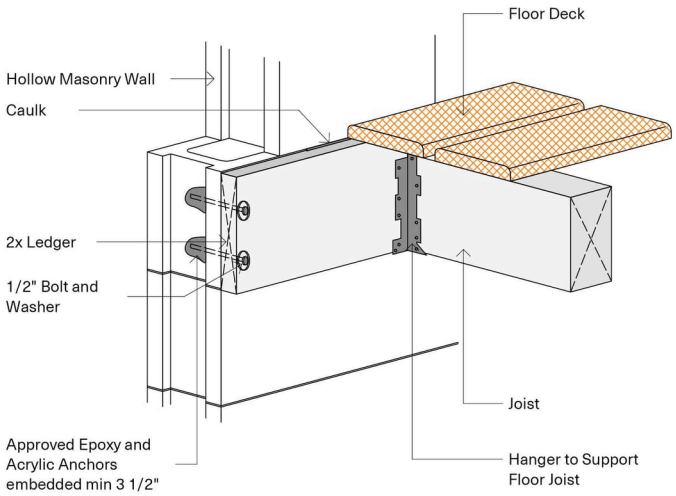


Figure 3.7 Details of joist attachments to a ledger beam

(This figure shows a timber ledger beam attached to a masonry wall with bolts. The detail also shows metal hangars providing support for timber floor beams.)

Hangers are commercially available for a variety of sizes of joists and loading conditions. Figure 3.8 shows a few examples of hangers manufactured by Simpson Strong-Tie®. Special types of anchors are designed for areas subject to hurricanes and seismic loadings. In the trade, these are known as “hurricane” and “seismic” hangers.

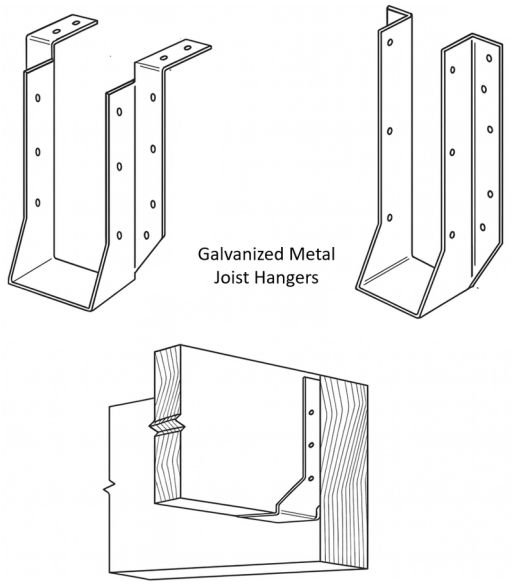


Figure 3.8 Examples of commercially-available joist hangers

(This figure shows examples of galvanized hangers and the detail of a wood joist supported by a hanger).

In situations when the reaction from floor and roof joists applies from one side of the masonry wall, such as the case shown in Figure 3.2, an out-of-plane bending moment develops. This bending moment is equal to $R \times e$, where R is the reaction and e is the eccentricity. If this moment is significant, the wall needs to be designed for this moment as well as for other loads, such as the axial load and shear. However, generally, if the resultant of all axial compression loads applied within the middle 1/3 of the cross section, i.e., when $e' \leq h/6$, where h is the wall thickness and e' is the eccentricity of the resultant of all compression forces), no tension stress develops in the wall cross section. This is an indication that the applied moment is not significant. The

proof for this concept is provided later in this chapter for reinforced concrete walls.

Illustrative example – Example 3.1 in Appendix A demonstrates the computations for the eccentricity e and e'.

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3.4: DESIGN REQUIREMENTS FOR REINFORCED CONCRETE WALLS

Design requirements for most load bearing reinforced concrete walls follow the provisions of the American Concrete Institute, see for example ACI (2014). These provisions are generally embodied in Chapter 11 of ACI (2014) and cover a variety of conditions including most bearing walls. Walls that sustain lateral soil pressure (retaining walls), prestressed walls and precast walls are covered elsewhere within the ACI code. The following materials are primarily applicable to the design of non-prestressed cast-in place walls. Although some of these materials also apply to tilt-up walls, these systems are covered elsewhere in this chapter.

Effective length - As expected, concrete bearing walls are primarily under the application of a series of concentrated loads that are mainly the reactions of floor and roof beams. According to ACI (2014), the horizontal length of a wall considered as effective for resisting each concentrated load is limited by the smaller of

- The center-to-center distance between loads; and,
- The bearing width plus four times the wall thickness

Furthermore, the effective horizontal length for bearing cannot extend beyond vertical wall joints unless design provides for means to transfer forces across the joint. This may include, for example, horizontal reinforcing bars placed at the vertical joints.

As an example, consider a 6-inch (152-millimeter) thick wall with concentrated loads spaced at every 18 inches (0.46 meters), the bearing width for most floor or roof joists is around 4 inches. Therefore, effective horizontal length is the smaller of 18.0 and 28.0 inches (i.e., 4.0+4×6.0 = 28 inches or 0.71 meters), which is 18.0 inches (or 0.46 meters). In this example, thus we can use 18 inches (0.46 meters) of the length of the wall under a concentrated load or one foot of the length of the load under a linear load of $q = R/1.5$ pounds per foot (or $R/0.46$ kilonewtons per meter), where R is the reaction of the floor or roof joint.

Minimum thickness – The minimum thickness for a concrete wall is taken as the 1/25 of the lesser of unsupported length and unsupported height of the wall as long as the wall is not part of the exterior basement or foundation. However, the thickness cannot be taken less than 4 inches (102 mm). As an example, let's take the unsupported height about the same as the height of say roof to the ground in a one-story building, about 14 feet (4.3 meters) in a given building. For an unsupported horizontal length of about 20 feet (6.1 meters), the minimum wall thickness will be 14.0×12/25 = 6.7 inches (170 millimeters), which is larger than the minimum of 4 inches (102 millimeters).

If the wall is part of the exterior basement or foundation of the building, then the minimum thickness is 7.5 inches (191 millimeters).

3.4.1: Simplified Wall Design

3.4.2: Wall Shear Resistance

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3.4.1: Simplified Wall Design

In general, the ACI requires a complete design of a wall for all possible forces including bending moments, developed as a result of eccentric or out-of-plane forces, axial loads, and in-plane as well as out-of-plane shear forces. When there is a possibility for slenderness effect, and this would generally be the case in tall, thin walls under very heavy axial load, the applied bending moment will need to be magnified. This is similar to the design process used in reinforced concrete columns. However, in most usual cases, the loading is limited to the floor and roof loads, which act as compression forces, along with in-plane shear forces, when the wall is acting as the shear wall also. The axial load due to compression forces from floor and roof loads is generally low; and if the resultant of all axial loads (P) applies within the 1/3 of the middle portion of the cross section of the wall, then ACI allows using a simplified design process based on the axial load only. Notice that when the resultant of all axial loads applies within the middle 1/3 of the cross section, the load eccentricity *e* will be less than $h/6$. This can very easily be shown by employing the equation used for the axial tension or compression stress on the wall cross section. As shown in Figure 3.9, for a wall section with a thickness h and effective length (ℓ_E) the stress on the cross section is

$$
f=\frac{P}{A} \pm \frac{P e}{S_X}
\]

The positive sign in this equation indicates compression stress, while the negative sign means tension. Furthermore, in this equation, $\ell_E \times h$ (cross-sectional area), and S_X= section modulus, which is the moment of inertia (I) divided by $h/2$, where $I = \frac{\ell_E h^3}{12}$ (moment of inertia). Equation 3.4.1.1 can then be written as

$$
f=\frac{P}{\ell_E h} \pm \frac{P e}{\left(\ell_E h^3 / 12\right) /(h / 2)}=\frac{P}{\ell_E h}(1 \pm \frac{6e}{h}).
\]

If the term $(1 \pm \frac{6e}{h})$ in Equation 3.4.1.2 becomes negative, there will be a tension stress developed in the section. Thus, to avoid any tension stress in the section, the eccentricity e will need to be less than $h/6$. This simply means that the point of application of the force P has to be within the middle 1/3 of the cross section.

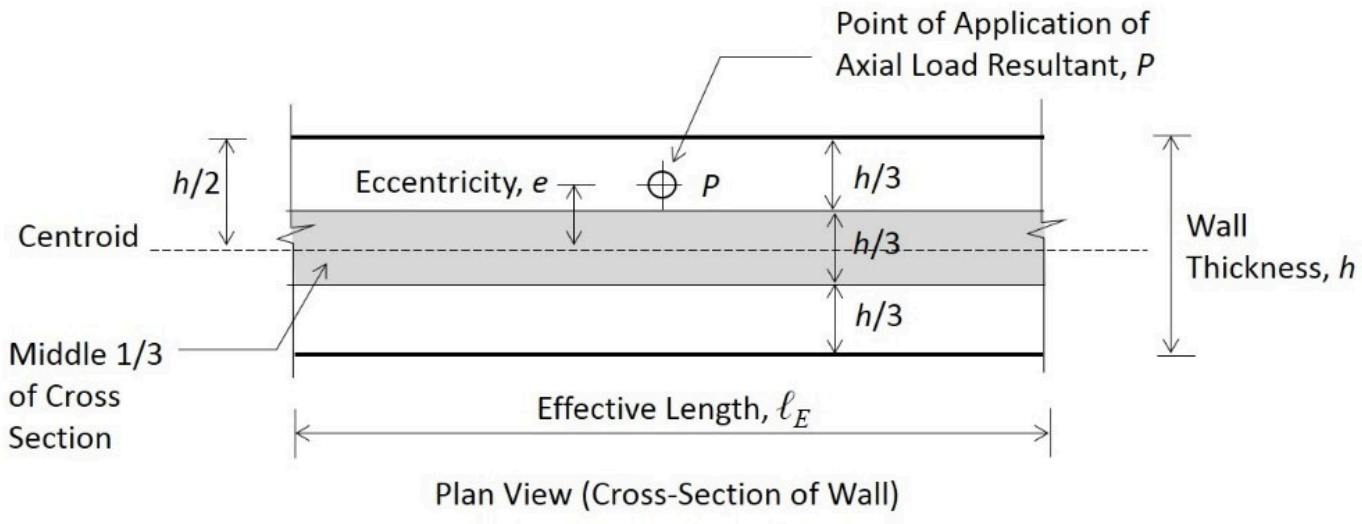


Figure 3.9 Cross section of wall with middle 1/3 shown in shaded area

(This figure shows the cross-section of a wall seen from above. The middle one-third core is shown as a shaded area. A load is shown outside of this area, which will cause tension stress.)

Using the simplified method, the nominal axial load resistance P_n is computed using the following equation, Eq. 11.5.3.1 of ACI (2014).

$$
P_n=0.55 f_c^{\prime} A_g\left[1-\left(\frac{k \ell_c}{32 h}\right)^2\right]
\]

In which

$f_c' =$ compressive strength of concrete in psi

$A_g =$ gross area of the wall (the product of the effective length and the wall thickness h) in square inches

$k =$ slenderness factor, which depends on the wall end conditions and lateral supports

$\ell_c =$ height of the wall in inches (please note that ACI indicates this parameter as the "length of compression member." The length refers to the dimension in the direction of the compression force, which in the case of the wall will be the height of the wall).

The value of the parameter k for most ordinary cases can conservatively be used as 1.0. However, Table 3.2 provides k for different boundary conditions as specified by ACI (2014).

Table 3.2 Values of k

Boundary Condition	k
Walls braced at top and bottom against lateral translation and restrained against rotation at one or both ends (top, bottom, or both)	0.8
Walls braced at top and bottom against lateral translation and unrestrained against rotation at both ends	1.0
Walls not braced against lateral translation	2.0

A proper design requires that

$$
P_u \leq \phi P_n
\]

In which, P_u is the applied total factored compression force over the effective length of the wall.

Illustrative example – Example 3.2 in the Appendix A demonstrates the ACI simplified procedure for computing the wall axial load resistance.

Wall reinforcement – Walls need to be properly reinforced to provide for (a) resistance in bending and compression; (b) resistance in shear; and (c) ductility. Depending on the type and forces applied, the reinforcement design may require detailed structural analyses following the ACI provisions. For example, in seismic areas, where walls are subject to in-plane and out-of-plane forces, the process is more involved and requires a detailed structural analysis to ascertain adequate reinforcement design and detailing. In most ordinary applications, when walls are primarily used as load bearing systems with a low to moderate in-plane shear force, the reinforcement design may only require a minimum amount as prescribed by the ACI code. In this section, the focus is primarily on this type of walls. In situations when a more detailed analysis and design is required, such as that in a seismic area or when the wall is also acting as a retaining wall, the recommendations of the ACI code will need to be reviewed and followed for proper reinforcement design.

One of the key parameters in deciding whether the minimum steel is adequate for the bearing wall is the factored applied in-plane shear force V_u. In most ordinary cases in low-rise applications, where the wall is also used as shear-resisting system (to resist against lateral forces from wind and earthquake effects), the value of V_u can easily be computed by following the recommendations of codes and standards such as the ASCE-7 (2022) and the IBC (2017). These documents provide basic information in regard to parameters that determine lateral forces based on (a) regions; (b) soil condition; (c) importance of the structure; (d) the severity of the intensity of the natural event causing lateral forces on the building; and (e) the distribution of the lateral force by floor levels.

The wall reinforcement is made up of both horizontal (transverse) and vertical (longitudinal) bars. The concrete cover for the protection of bars is specified as follows.

If the wall is exposed to weather or in contact with ground:

For #5 bar sizes and smaller, concrete cover; 1.5 inches (38.1 millimeters)

For #6 to #18 bars, concrete cover: 2 inches (51 millimeters)

If the wall is not exposed to weather and not in contact with ground:

Concrete cover: 0.75 inches (19.1 millimeters)

If the wall is cast against, and permanently exposed, to ground

Concrete cover: 3 inches (76.2 millimeters)

Note that the bar sizes specified follow the US customary system of units. And as such, a #5 bar has 5/8 inches of diameter, number 6 has 3/4 (i.e., 6/8) of an inch diameter and so on (see the supplements to this chapter for the table of bar sizes in US customary system of units and the equivalent "soft" metric sizes). With the aforementioned requirements, the concrete cover for a bearing wall can then be equal to 0.75 inches (19 millimeters) on the interior and 2 inches (51 millimeters) on the exterior side. Notice that these are minimum covers; and for simplicity, the cover can be taken as 2 inches (51 millimeters) on each side as long as it is not exceeding $h/3$, where h is the wall thickness.

The placement of horizontal and vertical bars is generally in the form of two grids (curtains), one in front and one in the back face. Exceptions to this requirement are for walls that (a) have a thickness less than 10 inches (250 millimeters); (b) are not used as retaining walls; and (c) are not basement walls.

Walls that meet the requirement on axial load eccentricity limit of ($e \leq h/6$) and the resistance given by Equation 3.4.1.3 can be designed for a minimum reinforcement as described below. These values are for non-prestressed walls. The minimum steels are

Vertical steel (longitudinal bars) in square inches per foot (or square millimeters per meter):

$$
A_{s v}=\rho_{\ell} b h
\]

Horizontal steel (transverse bars) in square inches per foot (or square millimeters per meter):

$$
A_{s h}=\rho_t b h
\]

(1) For cases where $V_u \leq 0.5 \phi V_c$

In which V_c is the shear resistance of the wall cross section (as explained later), ϕ is the shear resistance reduction factor (0.75), $b = 12$ inches (305 millimeters), and ρ_ℓ and ρ_t are the vertical and horizontal steel ratios, respectively. The values of the steel ratio ρ_ℓ (designated by ACI as longitudinal steel) and ρ_t (designated by ACI as transverse steel) are given from Table 3.3 for various conditions.

(2) For cases where $V_u > 0.5 \phi V_c$

The values of steel ratios must meet (a) and (b) given below.

(a) The value of ρ_ℓ is taken equal to at least the larger of the value calculated by Eq. 3.4.1.7 (as given below), which is Equation 11.6.2 of ACI (2014), and 0.0025, but need not exceed ρ_t in accordance with Table 3.3.

$$
\rho_{\ell} \geq 0.0025+0.5\left(2.5-h_{\mathrm{w}} / \ell_w\right)\left(\rho_t-0.0025\right)
\]

(b) The value of is ρ_t taken at least equal to 0.0025.

In Eq. 3.4.1.7, h_{w}/ℓ_w is the ratio of the wall height to wall length. For values of $h_{\mathrm{w}}/\ell_w \geq 2.5$, $\rho_\ell = 0.0025$; and for $h_{\mathrm{w}}/\ell_w \leq 0.5$, $\rho_\ell = \rho_t$

In practice, the steel is designated by the bar size and spacing. However, there are limits on the spacings as described below.

For vertical reinforcement:

For cast-in-place walls, the spacing $S_v \leq 3h$ and 18 inches (457 millimeters). However, if the in-plane shear force is sufficiently large that the wall requires shear reinforcement, the spacing of the vertical reinforcement also cannot exceed $\ell_w/3$, in which ℓ_w is the length of the wall that is resisting the in-plane shear force. For pre-cast walls, $S_v \leq 5h$ and 18 inches (457 millimeters) for interior walls and 30 inches (762 millimeters) for exterior walls. If shear reinforcement is required, because of a large in-plane shear force, then $S_v \leq 3h$ and 18 inches (457 millimeters) and $\ell_w/5$.

Table 3.3 Steel ratios ρ_ℓ (for vertical bars) and ρ_t (for horizontal bars) in walls when $V_u \leq 0.5\phi V_c$ (units are in square inches per foot)

Type of Wall	Type of Steel	Bar/Wire Size	f_y (pounds per square inch)	ρ_ℓ	ρ_t
Cast-in Place	Deformed Bars	$\leq$ #5	$\geq$ 60,000	0.0012	0.0020
			$<$ 60,000	0.0015	0.0025
		$>$ # 5	Any	0.0015	0.0025
	Wire	$\leq$ W31 or D31	Any	0.0012	0.0020
Pre-cast	Wire	Any	Any	0.0010	0.0010
Notes: These values are for non-prestressed walls. See the supplements to this chapter for bar sizes, diameters and areas.					

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3.4.2: Wall Shear Resistance

The ACI 318 uses the following equation for computing the shear resistance against in-plane shear forces when the applied load on the wall is in compression.

$$V_c = 2\lambda\sqrt{f'_c}hd \tag{3.4.2.1}$$

Where the parameter λ =1 for normal weight and 0.75 for light weight concrete. The dimension $d = 0.8\ell_W$ and is measured along the length of the wall. If the force on the wall or wall segment is in tension, V_c can conservatively be taken as zero. This condition usually does not happen in bearing walls in most ordinary situations. It may be a prevalent case when the wall is under a very small axial load and is subject to a very large potential lateral force from, for example, an earthquake.

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3.5: BEARING WALL DISTRESS CONDITIONS AND REPAIR METHODS

3.5.1: Distress in Masonry Walls
3.5.2: Distress in Concrete Walls

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3.5.1: Distress in Masonry Walls

Distress conditions in brick and CMU load bearing walls are in the form of cracks, displaced or damaged blocks, crushed blocks, and loss of mortars. Cracks in masonry load-bearing walls can be caused by such factors as (a) temperature variations; (b) differential foundation settlement; (c) heavy axial compression loads; (d) excessive in-plane shear forces; and (e) excessive out-of-plane forces, which cause bending moments. Temperature variations often result in nearly vertical cracks. This is prevalent when the wall is relatively long and there are no expansion joints provided. This would be similar to the situation discussed before regarding the lack of expansion joints in veneer walls. Expansion joints provided at about every 25 feet (7.6 meters) along the wall and 10 feet (3.1 meters) from the corner will practically eliminate this kind of cracks. A differential foundation settlement often results in diagonal cracks and in certain cases a cluster of short vertical cracks also. Heavy in-plane shear forces, which are usually the results of wind or earthquake loads, will also result in diagonal cracks. When heavy out-of-plane forces are applied, bending moments develop. If the wall is not properly designed and reinforced for these forces, horizontal cracks develop. Depending on the type of masonry blocks, mortar types and grout used, cracks may run along joints or through blocks as well. Figure 3.10 provides a summary of the types of cracks that can be observed in load-bearing masonry walls.

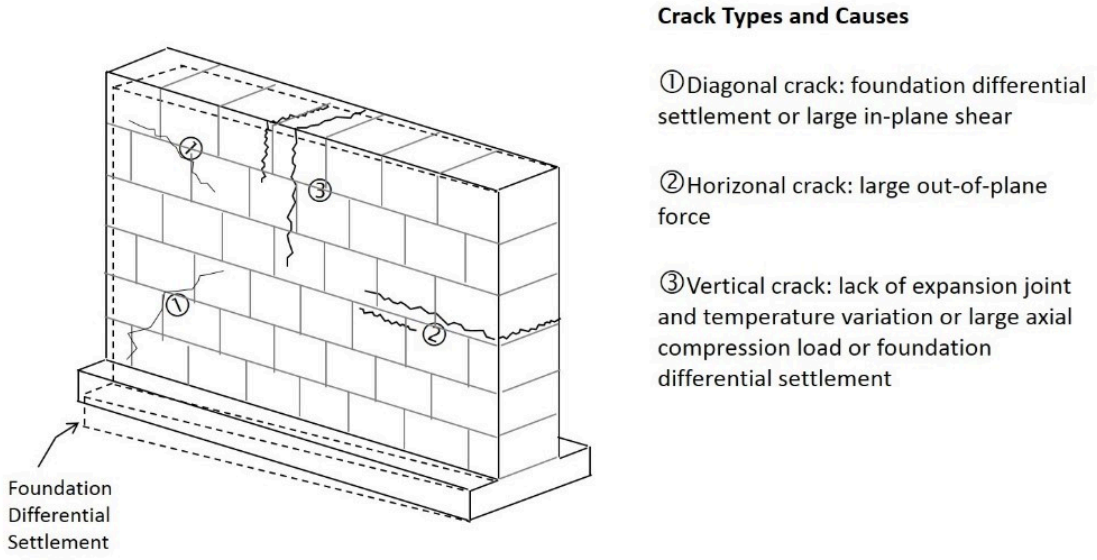


Figure 3.10 Crack types in a load-bearing masonry wall

(This figure shows three different kinds of cracks in a masonry wall. Two diagonal cracks are marked with number 1 and are because of foundation differential settlement or large in-plane shear forces. Two horizontal cracks marked with number 2 and are because of a large out-of-plane force. Two vertical cracks are marked with number 3 and are because of lack of expansion joints, temperature variations, or large axial loads.)

At times, a crack may be a result of more than one factor. Figure 3.11 shows a crack in a masonry load bearing wall. This crack, being vertical in upper and diagonal in lower part of the wall, is most likely the result of a combined heavy compression load on the wall and partial settlement of the foundation.

Small cracks of 1/8 inches (3.2 millimeters) in width can be repaired with mortar as long as the crack is stable and is not growing. If cracks are severe, the wall segment or the entire wall may have to be rebuilt with proper design and adequate reinforcement and grout as explained earlier. Furthermore, bond beams will need to be constructed as needed.

In situations when the wall needs to be reconstructed, we must also investigate the condition of the foundation. If the foundation is in good shape, still we need to compute the total load transmitted from the foundation to the soil to make sure it is adequate with consideration for the soil bearing capacity. The total pressure must be less than the allowable soil bearing capacity. Furthermore, the foundation will need to be sufficiently deep below the freeze line. If existing footing is utilized, dowel bars will need to be embedded (and secured using epoxy) in the footing. These dowels will need to be overlapped properly with vertical steel reinforcement. If the foundation is not adequate, a new one will need to be designed and constructed with proper size and

reinforcement.

Displaced, crushed and isolated cracked masonry blocks will need to be removed, area cleaned of debris and new blocks placed with mortar applied to repair an affected wall.



Figure 3.11 Crack in masonry

(This figure shows the photo of a diagonal crack in a brick wall.)

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3.5.2: Distress in Concrete Walls

The types of damage often observed in concrete walls are numerous and can happen because of one or more factors. In many situations, the damage may not be easily visible; and as such, some types of non-destructive test and/or evaluation or long-term monitoring may be necessary to find the cause and severity of a damaged wall. Most common damage cases are as described below.

Cracks – the severity of cracks is determined by measuring their width and whether they are still growing. Most concrete walls exhibit cracks. However, it is important to know what crack width is critical. The current ACI code does not directly address the thresholds levels for cracks. The limits summarized in Table 3.4 are from ACI 224R-01, ACI (2001).

*Table 3.4 Crack width limits, from ACI 224R-01**

Exposure Condition	Crack Width	
	Inches	millimeters
Dry air or protective membrane	0.016	0.41
Humidity, moist air, soil	0.012	0.30
Deicing chemicals	0.007	0.18
Seawater and seawater spray, wetting and drying	0.006	015
Water-retaining structures	0.004	0.10
* ACI (2001), reapproved, 2008.		

It is noted that the values listed in Table 3.4 are intended to be used as general guidelines. In certain specific cases, values different from these are recommended and can be used. For example, the National Ready Mix Association in their CIP-7 document specify 1/8 of an inch (3.1 millimeters) to be acceptable in most basement walls, which are generally unreinforced, CIP (2000).

Cracks in concrete walls can be caused by such factors as (a) temperature variations; (b) concrete creep and shrinkage; (c) differential foundation settlement; (d) heavy axial compression loads; (e) excessive in-plane shear forces; (f) excessive out-of-plane forces; (g) stress concentration at corners of openings; (h) corrosion of rebars; and (i) expansive aggregates used in making concrete. The crack appearance can be patchy, vertical, diagonal or horizontal. Generally, diagonal cracks can be attributed to a differential foundation settlement or a heavy in-plane shear force; while vertical cracks may be because of a heavy axial load, severe temperature variation or expansive aggregates. Horizontal cracks are most probably due to an out-of-plane force such as in cases where the concrete wall is under soil pressure. When rectangular openings are used in a concrete wall, corners are considered as regions of nonlinearity with a potential for stress concentrations. If corners are not properly reinforced, diagonal cracks may develop (see Figure 3.12).

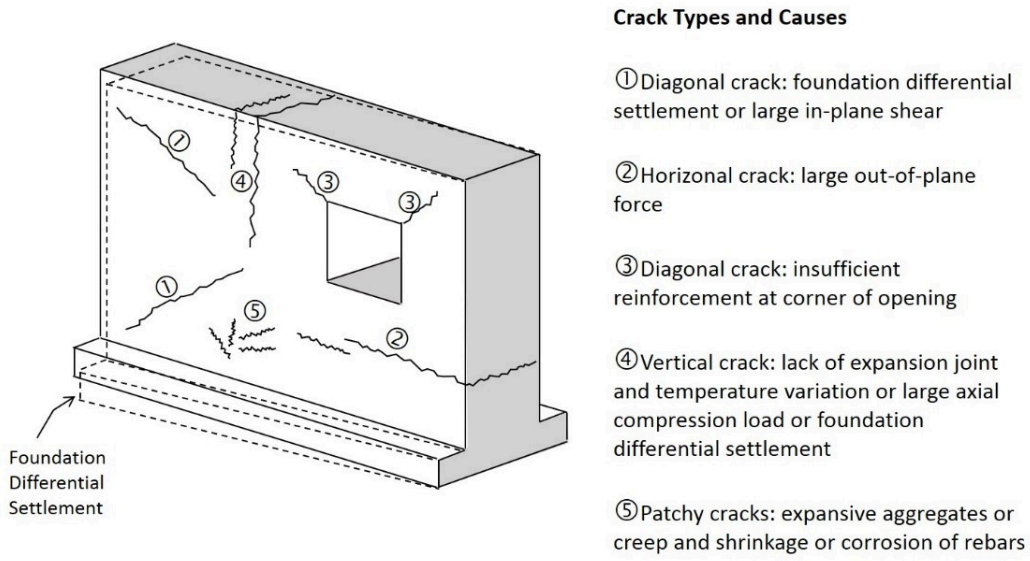


Figure 3.12 Crack types and potential causes in concrete walls

(This figure shows five different kinds of cracks in a masonry wall. Two diagonal cracks are marked with number 1 and are because of foundation differential settlement or large in-plane shear forces. Two horizontal cracks are marked with number 2 and are because of a large out-of-plane force. Several small diagonal cracks around an opening are marked with number 3 and are because of insufficient reinforcement at the corners of the opening. Two vertical cracks marked with number 4 and are because of lack of expansion joints, temperature variations, or large axial loads. Several patchy cracks are marked with number 5 and are because of expansive aggregates, creep and shrinkage or corrosion of rebars.)

Crack widths are measured using a simple crack comparator (which is a wallet size transparent ruler with lines of different thicknesses marked on it), a piano wire or more sophisticated devices, called crack meters. The repair method may simply involve filling the crack with epoxy, when the crack width is 1/8 inches (3.2 millimeters) or smaller. More severe cracks may require removal of portions or the entire wall, and rebuilding following proper design and detailing. In situations when wall is partially repaired, we must make sure the crack width is not increasing with time. This requires monitoring the crack width over an extended period of time to make sure the crack is not dynamic. One simple method is to use a patch of paster on a small portion of the crack area. If the crack is dynamic, after some time, the crack will appear on the plaster also. Another method is to use a crack meter, as described later in the chapter on non-destructive test and evaluation methods. If a crack is dynamic, it will need to be controlled before a repair is done. For example, in the case of foundation settlement, the foundation can be stabilized by using underpinning before any crack repair is done.

Concrete spalling – Concrete spalling is the loss of concrete cover. It happens in the form of patches where rebars become visible. It can happen because of (a) inadequate concrete cover; (b) corrosion of rebars (which expand and put pressure on concrete); (c) freeze and thaw cycles; and (d) poor workmanship during construction. In situations where the rebars are intact and without damage, the damaged area can be patched with quality concrete. More extensive damage, and especially when rebars are severely deteriorated, requires a more rigorous process for repair of both damaged concrete and steel. The following steps summarize what are generally needed to be followed in these situations.

- Remove as much concrete needed in the damaged area to reach sound concrete, continue removing additional concrete into sound concrete for a minimum of 1/4 inches (6.4 millimeters) and a maximum of 1 inch (25.4 millimeters).
- Clean and roughen surface; and apply bonding compound on old sound concrete. Note: some concrete repair experts believe bonding compound is not needed – it is a good idea to review the recommendation of the bonding compound manufacturer or consult with a concrete repair expert.
- Use stainless steel concrete wedge anchors type or stainless-steel pins - 4 per square foot with a minimum embedment length of 1½ inches (38.1 millimeters).
- Remove all loose and corroded and pitted steel, clean area – use new bars, if needed, overlapping with old ones, and apply a corrosion inhibiting agent to all exposed rebars. Alternatively, and in lieu of overlapping, mechanical couplers can be used to attach new reinforcing bars to the old ones. This is provided the thickness of the wall is adequate to accommodate installation of couplers. The ACI recommendations for using mechanical couplers will then need to be followed.
- Fill area after concrete removal and surface preparation with non-shrink or quick-set concrete or shotcrete.

Figure 3.13 shows an illustration of repair work using these five steps.

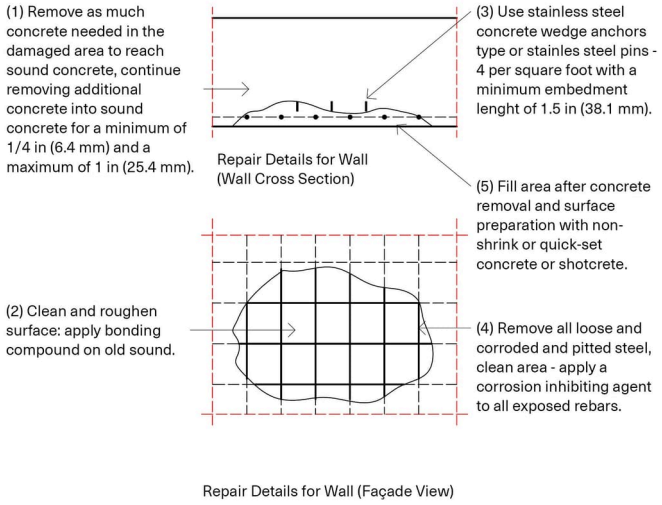


Fig. 3.13 Example of concrete spall repair work

(This figure shows two details. The one on the top shows the cross-section of a wall looking from above. It shows repair for concrete spalls. The detail on the bottom shows the spalls viewed on the surface of the wall. Five steps are explained for repair. Step 1 suggests removing damaged concrete to reach sound concrete. Step 2 suggests cleaning and roughening the area and adding a bonding agent. Step 3 suggests adding stainless steel anchors of about one and half inches in length (38.1 millimeters). Step 4 suggests removing all loose and corroded steel rebars and adding new rebars. Step 5 suggests filling the area with non-shrink or quick-set concrete or shotcrete.)

Poor workmanship – Poor workmanship may appear in the form of cracks, void in concrete or crumbled concrete. Furthermore, the required compressive strength may be compromised. Too little or too much vibration at the time of concrete work, lack of expansion joints, low quality aggregates, substandard concrete mix (inadequate water cement ratio, contaminated aggregates, water with too much salt content), removing forms too early, and inadequate forms to support wet concrete are examples of poor workmanship. It is important to have a good quality control in place at the time of repair work to reduce concrete damage because of these and other factors that are related to poor workmanship.

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3.6: TILT-UP PANELS USED AS FAÇADES

The term 'tilt-up' originated in the 1940s to describe panels or wall segments cast horizontally on a floor slab and then lifted into position. Each tilt-up panel is secured to adjacent panels via metal brackets and anchored to the foundation, creating a stable façade system. Typically load-bearing, tilt-up façades are widely used in industrial, manufacturing, and storage facilities, with recent applications extending to churches, schools, and other assembly buildings. These panels are engineered to support gravity loads and resist wind and seismic forces. Also known as 'tilt-wall' or 'site-cast precast concrete walls' in technical specifications, 'tilt-up' is the preferred, widely accepted term for this construction method.

Tilt-up panels are cast horizontally and then lifted into place to form the building façade and complete the structure. Sufficient reinforcement is incorporated to meet strength requirements during both the construction and lifting phases and to support applied loads after the panels are erected. In situations where a concrete bracket is integrated with a panel to support the roof girders, special reinforcement is needed per requirements of ACI, see for example ACI (2014). Additional reinforcement is added at critical points where regions of stress concentration are likely resulting in cracks. These locations are around openings and close to metal brackets that connect adjacent panels together. As with other concrete walls, the main reinforcement consists of a grid of vertical and horizontal rebars as illustrated in Figure 3.14.

Features – Tilt-up panels are generally handled only once. They are lifted or tilted from the casting slab and erected in their final position in one continuous operation. Panels are usually of large sizes and weights. And as such, they can only be constructed on site and in close proximity to their final locations. Being lead-bearing elements, panel gravity loads are supported directly by the wall and foundation instead of being supported by a structural frame system. If structural frames are also present, for example as part of an interior dedicated space such as an office in an industrial facility, tilt-up panels are generally erected before the structural frame is constructed (TCA, 2011).

Properties and design detail – Panel weights depend on the thickness, which varies between 5 to 8 inches (140 to 203 millimeters). With concrete weighing about 150 pounds per cubic foot (2,403 kilograms per cubic meter), so depending on the thickness and panel size the weight can be 80,000 pounds (36,288 kilograms) or more per panel for a typical height of about 38 feet (11.6 meters) and width of about 21 feet (6.4 meters), see Table 3.5. Since the panels are cast horizontally on a flat slab, usually formwork is only needed for the openings and the perimeter of each panel. The concrete curing time before panels can be lifted is about a week to 10 days, unless special conditions exist that need a longer curing time, for example in colder weather conditions.

Table 3.5 Total weight in pounds (kilograms) for panels of 38 feet (11.6 meters) height for different thicknesses in inches (millimeters)

Kind of Panel	5.5 (140)	6 (152)	6.5 (165)	7.25 (184)	8 (203)
18 feet (5.5 meters) wide	47,025 (21,331)	51,300 (23,270)	55,575 (25,209)	61,990 (28,119)	68,400 (31.026)
21 (6.4 meters) feet wide	54,865 (24,887)	59,850 (40,756)	64,840 (29,865)	72,340 (32,813)	79,800 (36,197)

Figure 3.14 illustrates an example of a tilt-up detail with its reinforcement. Notice the attachment of the panel to the foundation through pre-embedded dowel bars in the foundation. The dowel bars are aligned with pre-drilled holes in the panel and filled with epoxy as shown in Figure 3.14. Also, the floor slab is attached to the panel through dowel bars. The panel shown in Figure 3.14 has a concrete bracket to support the main girder in the roof.

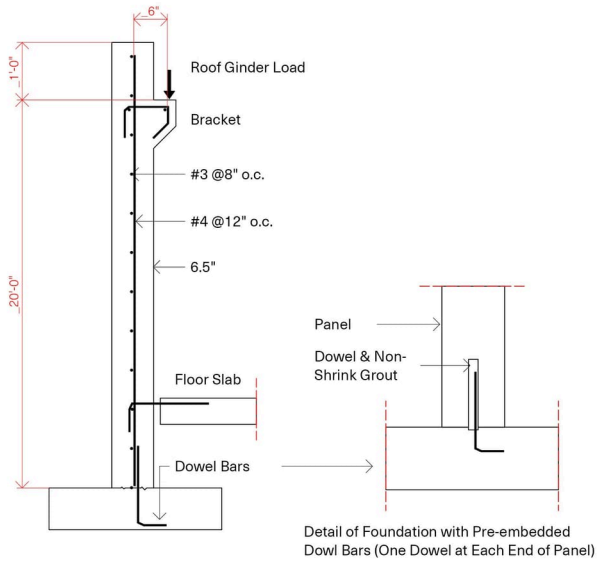


Fig. 3.14 A typical detail of a tilt-up panel reinforcement and attachment to the foundation and floor slab

(This figure shows the vertical cross-section of a tilt-up panel and its foundation. The reinforcing grid shown in the core is made up of number 4 vertical bars and number 3 horizontal bars. At the top, a concrete bracket is shown as a support for roof girders. To the right, the detail of dowel bars for the foundation is shown.)

Inner details – The details in Figure 3.14 mainly show the main panel reinforcement. However, in addition to the reinforcement, an insulation layer is also embedded within the concrete in each panel. The reinforcement is often in the form of a one-layer mesh made up of #3 or #4 bars. Panels are designed per requirements of the ACI and other pertinent codes, see ACI (2014) and ACI (2015).

Temporary braces – Braces are used to accommodate erecting the panels and maintaining their stability before the connecting brackets and roof are installed. At the time of casting panels, details for the attachment of braces are considered and properly designed and installed. Once panels are erected, the braces are removed. However, this will leave areas on the surface of the panels that need to be patched and smoothed out. Figure 3.15 shows a typical system with braces.

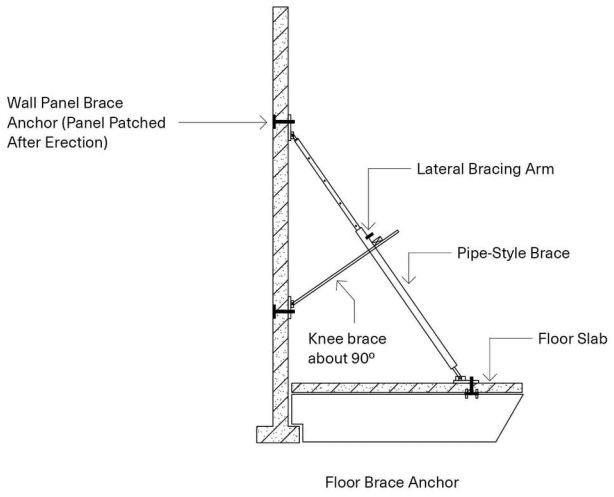


Figure 3.15 Bracing detail

(This figure shows the details of bracing used when lifting tilt-up panels to complete construction. A diagonal arm is shown

attached to the floor slab and a cross bracing attaching the arm to the base of the panel is also shown.)

Stages of construction – The following steps indicate the various stages in constructing and lifting panels to form the façades.

1. Panels are cast horizontally on a slab
2. Panels are lifted by cranes (they may weigh about 60 tons or more)
3. The erected panels are temporarily braced and typically connected to one another though steel brackets (shear connectors)
4. The roof system is completed with proper attachment to the walls after the panels are up, the braces are then removed
5. Joints are caulked (using appropriate caulking compound)
6. Wall finishes are applied to complete the building envelope
7. Architectural details are completed (see Figure 3.16)

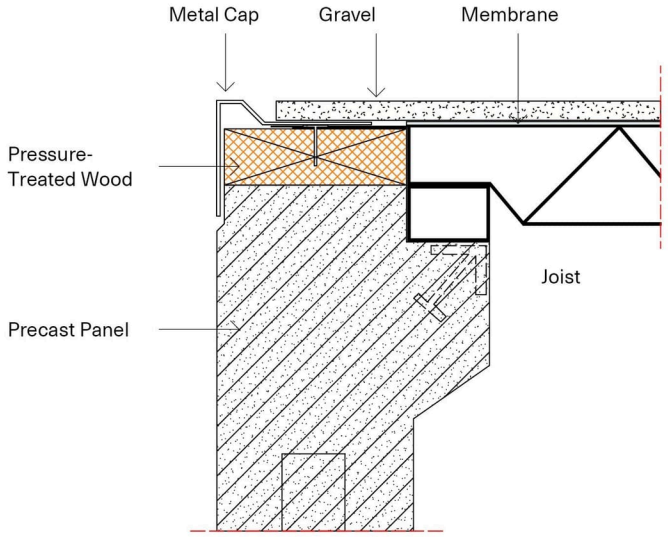


Figure 3.16 Architectural detail at roof connection to panel

(This figure shows the detail of how roof girders are supported by the concrete bracket at the top of a tilt-up panel. A partial detail of the roof including a membrane and gravel as well as a metal cap at the top of the wall is also shown.)

3.6.1: Distress Conditions in Tilt-up Panels

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3.6.1: Distress Conditions in Tilt-up Panels

Tilt-up panels experience similar distress conditions as those in concrete walls. These include cracks, material spalling, and reinforcement corrosion. In addition, because of their special way of construction and building assembly, there are also several additional distress conditions that pertain to connection of panels.

Generally, buildings made up of tilt-up panels require low maintenance and repair, with a majority of issues arising during construction or because of poor workmanship when performing repairs.

Cracks – The sources of cracks include foundation settlement, thermal expansion, shrinkage and stress concentration around openings and near the location of steel brackets that attach panels together. Another source of cracks includes flexural stresses developed in panels at the time they are lifted to form the structure. In situations when concrete brackets are attached to a panel to provide support for roof girders, such as the details shown in Figures 3.14 and 3.16, cracks may also develop in the bracket. The configuration of these cracks depends on such factors as the location of the reaction force applied on the bracket from the girders, the width of the bracket and its reinforcement details.

Minor cracks can be patched up and repaired; while if the cracks are extensive, and compromising the safety and the integrity of the structure, major repairs, including reconstruction may become necessary, which, of course, will be very costly.

Concrete spalling or chipped-away – Concrete spalling generally happens as a result of moisture penetration into the panel and/or corrosion of reinforcing bars. Minor damage areas may be patched and repaired, however, only after the source of the moisture penetration is identified and corrected. Moisture penetration can be the result of deteriorated caulking around windows and in between panels or poor workmanship when applying caulks. Poor air circulations inside the building may also cause moisture build-up and penetration into panels.

Chipped-away concrete can occur during panel erection or during repair and maintenance work. Damaged concrete can be patched and surface smoothed out provided the damage is not compromising safety and the integrity of the structure.

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3.8: Supplementary Materials to this Chapter

(1) ACI Load factors

1.4 D + 1.4 F

1.2 (D + F + T) + 1.6 (L + H) + 0.5 (Lr or S or R)

1.2 D + 1.6 (Lr or S or R) + (1.0 L or 0.8 W)

1.2 D + 1.6 W + 0.5 L + 1.0 (Lr or S or R)

1.2 D + 1.0 E + 1.0 L + 0.2 S

0.9 D + 1.6 W + 1.6 H

0.9 D + 1.0 E + 1.6 H

where,

D = dead load

E = earthquake load

F = fluid loads (defined level)

H = weight and lateral pressure of soil and water in soil

L = live load

Lr = roof live load

R = rain load

S = snow load

T = self-straining forces: contraction or expansion due to temperature changes, shrinkage, moisture changes, creep or differential settlements

W = Wind load

Flexure (bending) without axial load = 0.9*

Compression members = 0.65**

Compression members with spiral reinforcement = 0.70**

Shear and Torsion = 0.75

Shear for earthquake-resistant structures = 0.6

Bearing on concrete (direct pressure on concrete) = 0.65

* This is the maximum value, and depending on the strain at the farthest tension bars from the compression face, this value may be less. ** These are conservative values when compression controls the member.

(3) Bar sizes, diameters and areas

Table 3.5 Bar sizes, diameters and areas

US Customary Bar Size	"Soft" Metric Size	Weight (pounds per foot)	Mass (kilograms per meter)	Nominal Diameter (inches)	Nominal Diameter (millimeters)	Nominal Area (square inches)	Nominal Area (square millimeters)
#3	#10	0.376	0.561	0.375	9.525	0.11	71
#4	#13	0.668	0.996	0.500	12.7	0.2	129
#5	#16	1.043	1.556	0.625	15.875	0.31	200
#6	#19	1.502	2.24	0.750	19.05	0.44	284
#7	#22	2.044	3.049	0.875	22.225	0.6	387
#8	#25	2.67	3.982	1.000	25.4	0.79	509
#9	#29	3.4	5.071	1.128	28.65	1.00	645
#10	#32	4.303	6.418	1.27	32.26	1.27	819
#11	#36	5.313	7.924	1.41	35.81	1.56	1006
#14	#43	7.65	11.41	1.693	43	2.25	1452
#18	#57	13.6	20.284	2.257	57.33	4.00	2581

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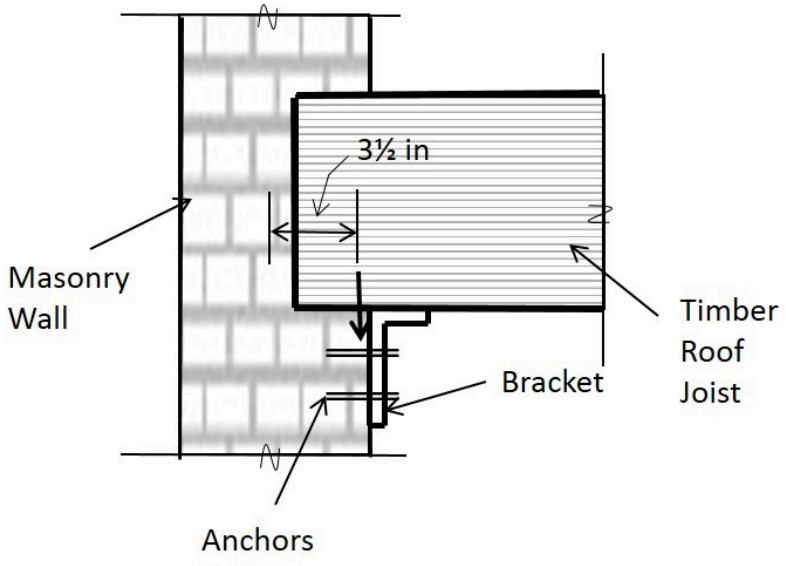
3.9: Exercises

E3.1 A masonry load-bearing wall is made up of CMU and is 8 inches (203 millimeters) thick. This wall is also used as a façade and is being investigated as part of a building façade inspection. The wall carries the load from the roof only with dead load and snow being the determining load combination. The roof joists each span 20 feet (6.1 meters) and rest on the wall. The factored applied load on each joist (using 1.2D and 1.6S) q_u is equal to 78 pounds per foot (1.15 kilonewtons per meter), where S indicates snow. The joists are made up of timber spaced at 18 inches (457 millimeters) on centers and are pocketed into the wall and a bracket used as a seat angle as shown in the figure. The eccentricity of the load transferred from each joist to the wall is estimated to be 3.5 inches, (89 millimeters) with respect to the centerline of the wall. The load from the masonry wall weigh is 85 pounds per square foot (4.1 kilonewtons per square meter). The height of this wall is 11.5 feet (3.5 meters) measured to its foundation.

a) Compute the factored self-weight of the wall with a dead load factor equal to 1.2, then compute the eccentricity of the resultant of all axial compression loads (P_u).

b) Based on the value you found in Part (a) compute the eccentricity of the resultant axial compression load and decide whether there will be any tension stress developed in the cross section of the wall.

c) Based on the values you found in Parts (a) and (b), compute the maximum compressive stress on the cross section of the wall.



(This figure shown a timber beam pocketed into a masonry wall with three and half inches of load eccentricity).

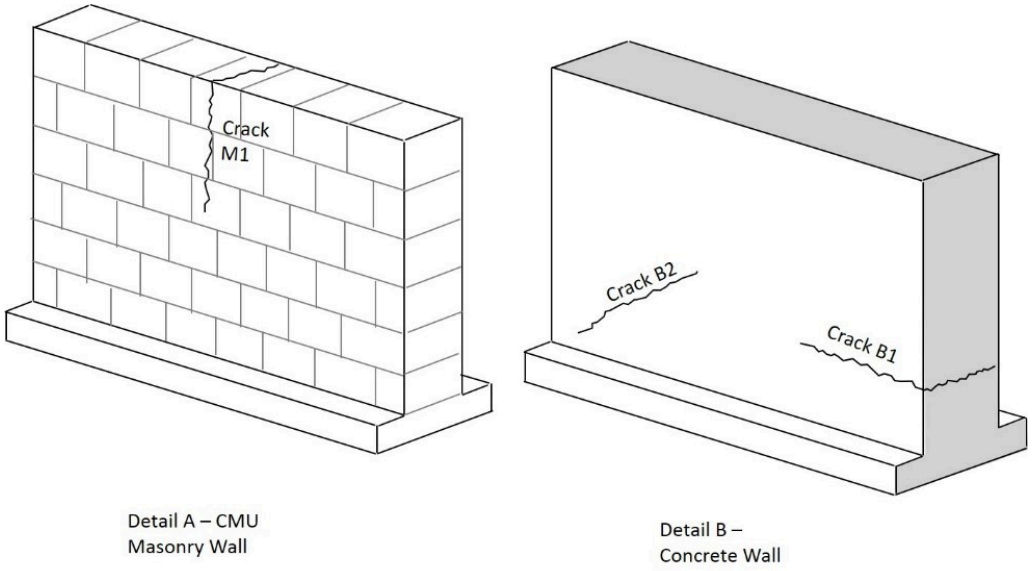
E3.2 A concrete façade is 8 inches thick (203 millimeters). Its length is 16 feet (4.9) and the height is 12 feet (3.7 meters). This wall is under gravity loads only. For reinforcement, #4 bars with a diameter of 0.5 inches (12.5 millimeters), are used with $f_y = 60000$ psi (413.7 megapascals). The concrete compressive strength $f'_c = 3,000$ psi (20.7 megapascals).

a) Determine the factored shear resistance of this wall.

b) Determine the vertical and horizontal steel that are needed (i.e, the spacing of #4 bars). Note that only one curtain of steel is used since the thickness is less than 10 inches (254 millimeters) and the wall is not a retaining wall.

c) Compute the factored axial load resistance ϕP_n, using the simplified equation, for this wall. Conduct the calculations for the unit (e.g., one foot) length of the wall.

E3.3 Examine the following two schematics showing locations and types of racks in two walls and describes the possible causes for Cracks M1, B1 and B2.



(This figure shows an illustration of two walls with cracks. The one on the left is a masonry wall and has a vertical crack marked as M1. The one on the right is a concrete wall with a horizontal crack marked as B1 and a diagonal crack marked as B2.)

E3.4 Which of the following statements is inaccurate?

a) The minimum reinforcement for a masonry load-bearing wall made up of CMU's is made up of vertical bars in every fourth cavity and horizontal bars in every other course.

b) Reinforced concrete bearing walls for which the eccentricity of the resultant compressions force is less than one sixth of the wall thickness can be designed based on the ACI simplified method.

c) Masonry load-bearing walls that are not subject to any in-plane or out-of-pane lateral loads (such as wind and earthquakes) can be built without reinforcement.

d) The limit for the spacing of vertical bars in reinforced concrete walls that are not subject to a design for shear is $3h$ or 18 inches (457 millimeters), whichever is smaller.

E3.5 Corrosion of rebars in concrete walls may result in concrete spalling off because the corroded metal:

a) Interacts with concrete.

b) Increases stress.

c) Expands in volume.

d) Loses cross section area.

E3.6 A reinforced masonry load bearing wall is 13.75 feet (4.2 meters) tall (from the foundation level) with an in-plane wind-induced shear load equal to 4,400 pounds (19.7 kilonewtons) acting at its top. The wall thickness is 10 inches (254 millimeters) and its length is 30 feet (9.1 meters). Compute the wall maximum axial load developed as a result of this shear force.

E3.7 The load computed for a masonry wall used in a one-story building are summarized below:

Axial dead load (including wall self-weight) is equal to 3,285 pounds per foot (47.9 kilonewtons per meter).

Axial live load from roof is equal to 2700 pounds per foot (39.4 kilonewtons per meter).

Axial snow load from roof is equal to 135 pounds per foot (2.0 kilonewtons per meter).

Axial load due to in-plane wind-induced shear is equal to plus or minus 390 pounds per foot (5.7 kilonewtons per meter).

Considering the ACI load factors and load combinations, compute the most critical value for the axial load for the wall.

3.8 Review items (i) - (iv) below and then select the most accurate answer regarding walls made up of tilt-up panels.

(i) The panels are generally handled only once. They are lifted or tilted from the casting slab and erected in their final position in one continuous operation.

(ii) The panels are usually of large size and weight. And as such, they can only be constructed on site and in close proximity to their final location in the structure.

(iii) Panel gravity loads are supported through structural frames and transferred to the foundation.

(iv) The panels are often designed for both the gravity and lateral loads.

a) Items (i), (ii), (iii) and (iv) are all correct.

b) Items (i), (ii) and (iv) are the only correct ones.

c) Only Items (i) and (ii) are the correct ones.

d) Items (i), (ii), (iii) and (iv) are all incorrect.

E3.9 Review the distress scenarios listed below identified by (i), (ii), (iii) and (iv). Then select the most accurate statement.

(i) Concrete chipped away and/or spalled off.

(ii) Caulk and joints between wall panels are worn out and damaged.

(iii) The shear connection brackets between panels are corroded or damaged or missing.

(iv) Concrete bracket supporting roof joists has signs of cracks.

a) These distresses are typically found in walls made up of wood panels.

b) These distresses may be seen typically in walls made up of tilt-up panels.

c) These distresses may be seen typically in walls made of unreinforced CMU's.

d) These distresses may be seen typically in walls made of unreinforced brick units.

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4: FAÇADES WITH TERRA-COTTA BLOCKS, STUCCO, AND STONE BLOCKS

Chapter Synopsis

This chapter covers façades with ornamental terra-cotta blocks, stucco, and stones; their damage scenarios and repair methods. Stucco or (render) is plaster-like exterior finish composed primarily of cement, sand, lime, and water. It is typically applied in multiple layers over a substrate such as metal lath, wire mesh, or masonry to create a solid, weather-resistant surface. Stone façades are typically used in combination with brickwork; however, in some older buildings, they may constitute the primary structural material for entire walls. Terra cotta blocks are made up of baked clays with glazing to look like stone. They have architectural appeal and have their own unique kinds of damage issues, as covered in this chapter. Most of the materials presented in this chapter on terra-cotta blocks, their applications, types, and damage scenarios are abridged from de Teel Patterson Tiller (1979).

4.1: Terra-Cotta

4.1.1: Types of Terra-Cotta

4.1.2: Characteristics of Glazed Architectural Terra-Cotta

4.1.3: Common Deterioration Problems

4.1.4: Repair Methods

4.2: Stone Façades

4.2.1: Popular stones in façade applications

4.2.2: Stone characteristics and properties

4.2.3: Distress conditions

4.3: Stucco

4.3.1: Construction process

4.3.2: Distress Conditions

4.4: Synthetic Stucco

4.4.1: Distress conditions

4.5: Chapter References

4.6: Exercises

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4.1: TERRA-COTTA

Glazed architectural terra-cotta was instrumental in the evolution of U.S. architecture, particularly within the Chicago School, Highrise, and Beaux Arts styles. Extensively used from the late 19th century until the 1930s, this material offered a modular, adaptable, and cost-efficient method for constructing walls and floors, especially for facades requiring intricate ornamentation. Although its popularity waned by the mid-20th century, numerous older buildings still feature terra-cotta facades. Consequently, the restoration and preservation of these elements present significant challenges for architects and engineers. de Teel Patterson Tiller (1979) provides methodologies for evaluating material degradation and offers best practices for the maintenance, repair, and replacement of deteriorated terra-cotta components. Additional information to terra-cotta use in buildings and their history can be found in McIntyre (1929) and Prudon (1976 and 1978).

In Italian, the term "terra-cotta" translates to "baked clay." In architectural applications, it refers to a premium grade of weathered or aged clay mixed with sand or pulverized fired clay, molded, and kiln-fired at high temperatures to achieve optimal hardness and density. The color of terra-cotta varies geographically, ranging from red and brown to white. Typically, terra-cotta is hollow-cast into blocks with open backs, resembling boxes, and reinforced internally with stiffening elements known as webbing. This webbing substantially enhances the load-bearing capacity of the hollow blocks without significantly increasing their weight.

The surface of terra-cotta blocks is finished with a glaze, which involves applying a slip glaze (a clay wash) or an aqueous solution of metal salts onto the air-dried blocks prior to firing. The glaze is architecturally appealing giving the façade appearance of those made with polished stones. This glazing process alters the block's color and creates a relatively impervious exterior surface. When properly maintained, and not damaged, the glaze provides excellent weather resistance, a rich coloration, and a durable surface that is highly resistant to chipping. However, as explained later in this chapter, this water-proofing property has not been adequate to entirely protect façades from moisture damage in situations where proper flashing, weep holes, or wick ropes were missing.

4.1.1: Types of Terra-Cotta

4.1.2: Characteristics of Glazed Architectural Terra-Cotta

4.1.3: Common Deterioration Problems

4.1.4: Repair Methods

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4.1.1: Types of Terra-Cotta

Terra-cotta types used in buildings can be categorized in four groups. These are:

- Brownstone terra-cotta
- Fireproof construction terra-cotta
- Ceramic veneer and cladding
- Glazed architectural terra-cotta

Brownstone terra-cotta, one of the earliest forms used in American architecture, emerged in the mid to late 19th century. Characterized by its dark red or brown coloration, brownstone terra-cotta is available in both glazed (typically with a slip glaze) and unglazed varieties. It was typically hollow cast and often used in combination with other masonry materials, imitating sandstone, brick, or natural brownstone. This material is commonly found in the works of architects such as Richard Upjohn, James Renwick, and Henry Hobson Richardson, and is associated with the Gothic and Romanesque Revival architectural movements. Its use is particularly evident in ornamental elements like moldings, finials, and capitals.

Fireproof construction terra-cotta was developed extensively in response to the rise of highrise construction in the United States. These hollow, rough-finished blocks were cost-effective, lightweight, and fireproof, making them ideal for spanning joists (in the form of I-sections) in floors, walls, and ceilings. While certain varieties remain in production, fireproof construction terra-cotta is no longer widely utilized in modern building practices.

Ceramic veneer and claddings, developed in the 1930s, remain widely used in contemporary construction. Unlike traditional architectural terra-cotta, ceramic veneer is not hollow-cast. Instead, it consists of a thin veneer of glazed ceramic tile with a ribbed backside, similar to bathroom tiles. Typically, it is attached to a grid of metal ties, which are anchored to the building structure.

Glazed architectural terra-cotta represents one of the most complex developments in masonry materials in the United States. These hollow units were either hand-cast in molds or carved in clay, heavily glazed (often to mimic stone), and then fired. Sometimes referred to as "architectural ceramics," glazed architectural terra-cotta was refined throughout the early 20th century and is closely associated with prominent architects such as Cass Gilbert, Louis Sullivan, and Daniel H. Burnham. Notable examples of its use include the Woolworth Building (built in 1913) in New York City and the Wrigley Building (built in 1921) in Chicago.

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4.1.2: Characteristics of Glazed Architectural Terra-Cotta

Glazed architectural terra-cotta shares many material properties with brick and stone, but it also exhibits key differences that must be understood when conducting façade inspections. It is also noted that glazed terra-cotta blocks are often blended with bricks and stones in façades in the form of cornices, ornamental pieces and moldings around windows. Thus, it is imperative that these differences are fully understood. These differences, as outlined below, are crucial for evaluating the material characteristics of glazed architectural terra-cotta and decisions on correctly identifying the damage types, especially when suggesting a repair method.

Identification of type – Glazed architectural terra-cotta is commonly found in both historic and contemporary buildings. Its appearance can closely resemble various types of stone. This similarity contributed to its popularity in early 20th-century façades, especially in ornamental features and cornices. Differences in glazing needs to be determined and are generally distinguishable by experts. The glazing can give terra-cotta blocks the look of limestone, granite, or other stones; and as such, unless examined by experts and with close inspection, these blocks can be mistaken for natural stone.

Application – Glazed architectural terra-cotta has been employed in two distinct types of building envelopes. The first is traditional load-bearing masonry walls, typically seen in medium-rise structures. The second is non-structural veneer walls, primarily in medium- to high-rise buildings. These two systems differ significantly in terms of damage scenarios. For veneer applications, the blocks are attached to the building via metal anchoring systems, either to a masonry back-up wall or a structural frame. In this case, a common source of damage stems from corrosion or inadequate design of the metal anchors. In load-bearing masonry walls, deterioration is often due to physical damage to the blocks, such as chipping or cracking.

Variety of deterioration and damage scenarios – Deterioration in glazed terra-cotta varies widely depending on its application. While damage to a single façade may be consistent across blocks, the type of damage can differ dramatically between applications and in one building versus another. In veneer systems, damage often results from corroded metal anchors, as mentioned previously. Other common issues include poor construction, cracks, mortar loss, damaged glazing, and broken or chipped blocks. These damage patterns are influenced by various factors, which often vary between buildings.

Inadequate original design and detailing – Since many terra-cotta installations are over a century old, their original design and detailing often fall short of modern expectations for façade integrity and durability. Glazed terra-cotta was once considered an effective waterproofing material for façades, which led to the omission of crucial moisture management details such as flashing, weep holes, and wicks. This lack of moisture control has contributed to significant water-related damage in terra-cotta blocks and their deterioration over time.

Costly repairs – Today, the production of terra-cotta blocks is limited, making the restoration of damaged façades expensive. Replacement blocks must match the originals in shape, color, and long-term durability. While a few manufacturers can produce custom replicas, the cost of these replacements is substantial. In some cases, synthetic materials are used to replicate the original blocks at a reduced cost. However, these materials can change in color and appearance over time, potentially leading to higher long-term maintenance costs.

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4.1.3: Common Deterioration Problems

Each façade inspection project involving terra-cotta blocks must be treated as a unique case due to the numerous variables, including material composition, original manufacturing processes, inconsistencies in installation, the number of component parts, ongoing repairs, and the diverse types and sources of deterioration. However, despite these variations, certain commonalities exist across different types of glazed architectural terra-cotta. As a result, some general observations can be made regarding the typical patterns of deterioration in this material. At times, the complexity of the deterioration may require hiring the services of an expert.

Material failure can most commonly be attributed to water-related problems. However, less frequent though no less severe causes may include: faulty original craftsmanship, which is often cited but hard to determine, stress-related deterioration, damage caused by later alterations and additions, or inappropriate repairs.

Water-related damage is a principal source of deterioration in glazed architectural terra-cotta. Terra-cotta systems are highly susceptible to such water-related deterioration problems as glaze crazing, glaze spalling and material loss, missing masonry units, and deteriorated metal anchoring, among others.

Crazing refers to the formation of small random cracks in the glaze, a common form of water-related deterioration in glazed architectural terra-cotta units (see Figure 4.1). When a new terra-cotta unit is first removed from the kiln after firing, it shrinks to its smallest possible size. Over time, the unit absorbs moisture from the air, causing it to expand – a process that can continue for many years. The glaze, however, has a lower capacity for expansion compared to the porous tile body of the terra-cotta. As the unit expands, the glaze is subjected to increasing tensile stress unable to cope with the expansion of the tile body. Eventually, if the glaze's tensile strength is exceeded, it will crack, leading to the condition known as crazing. Unless the cracks visibly extend into the porous tile body beneath the glaze, crazing should not be regarded as highly serious material failure. It does, however, tend to increase the water absorption capability of the terra-cotta unit, which may lead to more serious damage over time.

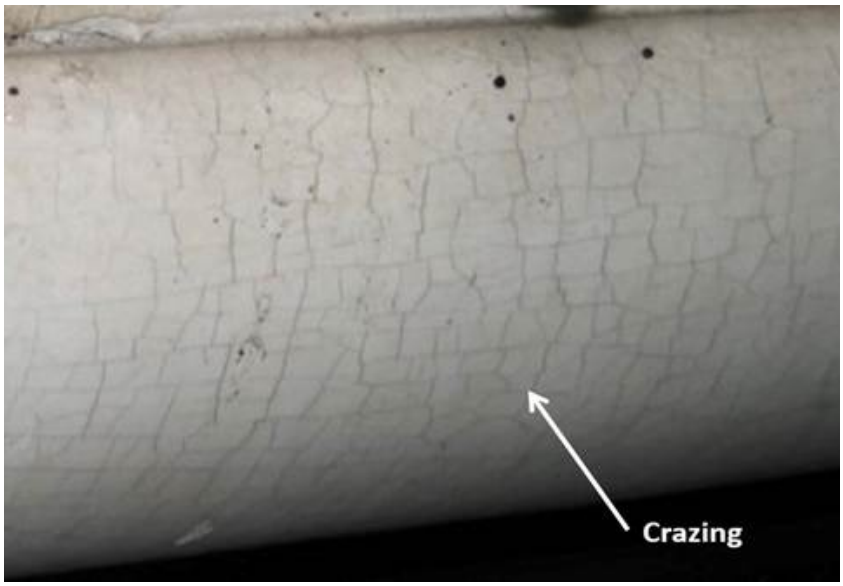


Figure 4.1 Terra-cotta crazing (from personal collection of the lead author)

(This figure shows a close-up photograph of patchy surface cracks known as crazing on terra-cotta blocks.)

Figure 4.2 shows an example of a stress-related crack in a terra-cotta block (as opposed to the crazing cracks shown in Figure 4.1).

Spalling refers to the partial loss of masonry material and is primarily caused by water, whether from ambient moisture or water trapped within the masonry system. In terra-cotta blocks, trapped moisture is a more common issue. This is often due to poor detailing in the original design, insufficient maintenance, or the lack of proper flashing and moisture management systems. In

most masonry walls, when no coating or glazing exists to act as a moisture barrier, trapped water tends to migrate outward through the masonry and evaporate over time. However, in terra-cotta façade units, the glaze acts as a barrier, preventing the moisture from escaping. As the water accumulates, it creates internal pressure that can cause sections of the glaze to spall off. This process is particularly accelerated by significant fluctuations in temperature.



Figure 4.2 An example of a stress-related crack in terra-cotta with crazing cracks are also visible (from personal collection of the lead author)

(This figure shows a photograph of a vertical crack in a terra-cotta block.)

Glaze spalling may appear as small coin-size blisters where the glaze has ruptured and exposed the porous tile body beneath. This may occur as several spots on the surface or, in more advanced cases of deterioration, it may result in complete loss of the glaze. Spalling of the glaze may also be the results of deterioration of the internal metal anchoring system because of corrosion (as explained earlier in Section 4.1.2). The increase in volume of the metal created by rusting creates increased internal pressures in the terra-cotta unit which, in turn, may spall the glaze, or in more extreme cases, cause material spalling. When material spalling occurs, the damage will further extend to the entire unit and further exposes it to moisture damage to the webbing and the metal anchors. This kind of damage alters the architectural appearance of the façade. Generally, the line of action in this situation is the replacement of the affected terra-cotta units.

Deterioration of mortar and adjacent materials is a common issue in façades constructed with terra-cotta units. High-quality mortar serves as the first line of defense against damage to terra-cotta façades. When mortar joints deteriorate, they become major entry points for water infiltration into the façade. This deterioration can result from poor craftsmanship during installation, exposure to airborne and waterborne pollutants, and neglect and insufficient periodic inspections and maintenance. The combination of deteriorated mortar, along with inadequate installation of flashing, copping stones, cornices, roofing, and improper caulking around windows and doors, can lead to significant damage to terra-cotta façades.

Stress-induced deterioration of glazed architectural terra-cotta is frequently observed in highrise buildings. The development of stress-relieving features, such as flexible joints and shelf angles, came relatively late in American building practices. As a result, most early highrise buildings (circa 1900–1920s), which were continuously clad in terra-cotta, were designed without sufficient provisions to accommodate material movement or structural shifts. This lack of flexibility often leads to large stress-related cracks or material degradation, caused by heavy loads, thermal expansion and contraction, or moisture absorption by the terra-cotta units themselves. Cracks that extend through multiple units are often indicative of stress-related issues, which further allow water to penetrate the terra-cotta system and exacerbate the damage.

Inadequate repairs commonly occur when substitute materials are used in place of authentic terra-cotta blocks. This is often the result of cost constraints or the difficulty of sourcing replacement terra-cotta units. It is not uncommon to see repairs where cracks are merely filled with caulk, a method that not only diminishes the architectural integrity of the façade but also accelerates further deterioration (see Figure 2.14). The caulk eventually degrades, allowing more water infiltration, which leads to additional damage to adjacent terra-cotta blocks. In some cases, repairs require replacing the anchors behind the units, a costly process.

When property owners seek to avoid these expenses, they often opt for cosmetic repairs, which do little to extend the life, and preserve the architectural aesthetic, of the façade.

Alteration damage typically occurs when additions such as signs, screens, or bird-deterrent systems are installed. These installations often require drilling holes or cutting into the terra-cotta units to secure the attachments to the building's frame or the backup wall. Over time, as caulking or anchoring materials deteriorate or the additional elements are removed during subsequent renovations, the resulting holes become significant sources of water infiltration, further compromising the integrity of the façade.

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4.1.4: Repair Methods

A cursory examination of terra-cotta façades is often insufficient to fully assess their integrity. In many cases, it may be necessary to remove certain units to reveal hidden damage and evaluate the condition of the metal anchors. Given the high cost of replacing terra-cotta units, it is crucial to perform a comprehensive inspection before deciding on the appropriate repair strategy. When water-related damage is discovered, repairs should only proceed after the sources of water infiltration have been identified and eliminated to prevent future issues. This typically involves the proper installation of new flashing or weep holes, guided by well-designed architectural detailing.

In cases where stress-related problems compromise the façade's overall integrity, such as long cracks running through mortar joints or even through the terra-cotta units themselves, the damage is structural. This may require a complete redesign, potentially involving the installation of shelf angles, relieving joints, or flexible joints. The services of an expert are necessary to design and implement appropriate structural elements and detailing.

Routine maintenance and repairs must be carried out carefully to avoid further damage to terra-cotta units. The following sections outline the key practices to follow, as well as those to avoid, during routine terra-cotta façade maintenance.

Terra-cotta cleaning – An approach to conduct a successful cleaning of terra-cotta façades involves removing excessive soil from the glazed surface without damaging the masonry unit itself. There are many cleaning agents commercially available. However, in most situations, a solution of water and detergent used along with a natural or nylon bristle brush will be sufficient. When this solution is not adequate to remove a more severe case of contamination, the affected terra-cotta units can be cleaned with steam or a weak solution of muriatic or oxalic acid. It is noted that any acid, in higher concentrations, may themselves become harmful by releasing salts within the masonry system, causing efflorescence deposits on the façade. Because of the sensitivity of the glazing, care must be exercised to avoid any kind of sandblasting or using abrasive brushes when cleaning terra-cotta units.

Waterproofing – Although the glazing is generally thought of acting as a moisture barrier on terra-cotta blacks, it is not exactly known whether the microcracks on the surface counteract the moisture-barrier capability of the glaze and allow moisture to penetrate the block. Based on observations in historical buildings, most crazing on historic glazed terra-cotta does not substantially increase the flow of moisture into the porous tile body compared to new material. However, when crazing is excessive, it is generally known that the glazing will not substantially prevent moisture penetration. It is also noted that the infiltration of moisture into the façade may be because of a host of other reasons, such as deteriorated caulking and lack of proper flashing and weep holes. Thus, again, thorough investigation in terra-cotta façades is a very crucial strategy to positively pinpoint to the water and moisture-related damage. The limited waterproof capability of the glaze at times may prevent the outflow of moisture; and as such, the moisture infiltrates through mortar joints to evaporate. In situation that joints are completely sealed, the trapped moisture may cause damage including the corrosion of the metal anchors.

Tuckpointing – Tuckpointing and repointing of damaged mortar is often carried out as needed to preserve the integrity of the façade. It is advised that the mortar selected for restoration has a lower compressive strength than the surrounding terra-cotta blocks. It is also noted that using Portland cement or coarsely screened mortars may cause point loading and/or prevent the outward migration of the water through the mortar joints, causing damage to terra-cotta blocks. Using waterproofing caulking compounds for repair of joints is a poor restoration practice that should be avoided or corrected if already applied. These compounds work as sealant trapping moisture inside causing further damage as explained earlier.

Repair of glaze spalling – Glaze spalling not only compromises architectural aesthetics but also becomes a segway for moisture migration inside, causing further damage. It is also a highly culpable source of water-related deterioration in terra-cotta. The practice for repair requires removing all loose material taking care not to damage the affected blocks. Proper coating will then need to be applied over the cleaned area. It is noted that finding a glazing material that will be long lasting may be difficult. And as such, several acrylic-based commercially-available products can be used to protect these exposed areas with a suitable color to match that of the glaze. However, usually these compounds have a limited life and require reapplication in about five years.

Material spalling repair – In situations when the extent of damage to a terra-cotta block is severe, the block will need to be removed and replaced with a new like one. This is usually the case when a unit appears damaged by visual inspection. Since at times finding the replacement pieces may take time, it is imperative to provide a temporary replacement to avoid water infiltration and other types of damage, such as changes in pressure points on blocks in the vicinity of those that have been removed. Temporary tuckpointing, the removal and saving of undamaged units to be used later, or the temporary installation of brick infill

to retard further deterioration may also become necessary. If the damage is not major and is limited to a few spalling areas, some on-site repair can be done. This kind of repair involves cleaning the area, carefully removing loose materials, applying a suitable compound compatible with the original materials, and then finishing the repair by applying a coat. This kind of damage can also involve smoothing the chipped areas and then applying the coat. Most terra-cotta experts advise against patching and against using cementitious and stucco-like materials. Since these materials will not properly bond with the terra-cotta materials, and furthermore, the difference in the coefficients of thermal expansion of the two materials (i.e., the repair materials and the original terra-cotta material) impedes a reliable attachment.

Repair of holes, grooves, and cracks – Holes, grooves, and cracks can be repaired using a carefully-selected sealant. Manufacturers are often the best source of advice on the type, color to match that of the damaged terra-cotta blocks, waterproofing capability, and suitability of suitable compounds.

Considerations for terra-cotta replacement – As indicated earlier, severely damaged terra-cotta blocks are generally required to be replaced. The following is a list of several notes in regard to the replacement practice.

- Replacing terra-cotta units needs to be done carefully as to not damage other blocks. Furthermore, all of the original deteriorated material should be completely removed.
- Preferably, replacement units should be anchored in a manner similar to the original. New metal anchors are available in a variety of shapes for different applications.
- The preferred method for replacement is to use in-kind units. This is costly since only a few manufacturers currently produce replicas of units on a custom-made basis. New terra-cotta units are usually machine made. Thus, the porous tile body of the new material tends to be more uniform but less dense and often not as durable. The glaze on the new units tends to be thinner than that on the older material and subsequently more brittle.
- Often the decision for repair includes replacing damaged ornamental work with simpler, less-costly and more readily available units. This approach is generally not acceptable since the finished product often loses its architectural aesthetics and charm.
- Depending on the kind of building, stone may be a suitable replacement material for damaged terra-cotta units. Major decision-making factors in selecting a stone block as a replacement include its heavy weight, difficulty in anchoring, and cost, since a color match and crafting the exact shape as the original block require specialized craftsmanship.
- In certain repairs, fiberglass blocks are considered for replacement. This is especially true in situations when replicas of special ornamental pieces are needed. In current manufacturing practice, a casting from an original intact piece is made for making copies with accommodation for anchoring included in casting. Major problems with fiberglass replacement are (1) color match, (2) fire code compliance, and (3) poor weathering and aging. Painting and coating used to match the original color is often unsatisfactory, as it discolors at a rate different than that of the historic glazed original units. Commercial products available include TerraGlas® (www.terraglas.com), for example.
- Using precast concrete units is yet another alternative in replacing terra-cotta units. Precast units can be cast hollow, use lightweight concrete with accommodation for using metal anchoring. These blocks are usually painted to match the terra-cotta original color, although concrete itself can also be tinted for color compatibility. The production is generally straightforward with a favorable cost.
- Fiber glass reinforced concrete is another material that can be molded to produce the replica of original pieces. The color changes over time, and being affected by weathering, may become an issue in the long run.
- Usually, any newly-manufactured block, whether stone, concrete, or fiberglass, must be re-anchored to the structural system or the backup masonry wall. The new replacement unit with anchoring requires to be embedded into the existing masonry backup wall by drilling a hole or slot for anchor using mortars or epoxy materials for proper bond strength.
- Use of new technologies includes 3D printing of pieces from synthetic materials from drawings that are prepared from digitization of photographs of intact pieces.

Table 4.1 summarizes the aforementioned common distress conditions in terra-cotta units, the damage appearance, causes, and potential lines of action to address each distress condition.

Table 4.1 Common distress conditions in terra-cotta units – causes and lines of actions

Distress Condition	Damage Characteristics	Causes	Line

Damaged, cracked and broken units; material spalling	Cracks in units; pieces chipped away	Heavy load; poor workmanship; poor handling in previous repairs; weathering	Minor cracks and missing quality sealant. Major damage needs repl
Crazing	Small cracks in the glaze with random pattern	Weather effects; temperature effects during unit cooling	No actions needed, if crac porous tile body beneath When cracks have extend units need to be replaced
Glaze damage and spalling	Small coin-size blisters appearing on surface	Weathering; deterioration of metal anchoring because of corrosion	Minor damage can be rep applying protective coat. Major damage requires u
Metal anchor failure	Loose, missing and broken units; rust stain appearing on surface	Faulty design, weathering; moisture migration into units	(1) Metal anchoring replac can be reused; (3) Damag
Mortar deterioration	Opening between blocks; unit dislocation	Poor workmanship; use of improper mortar in previous repairs; exposure to air- and water-borne pollutants; neglect	Minor damage can be rep place and applying prope Major damage requires re
Discoloration	Units different in color than other parts of façades	Improper prior repairs; poor selection of replacement for previously-damaged units	The affected units need t units.

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4.2: STONE FAÇADES

The use of stone as an exterior façade element has been historically popular due to its ability to be fabricated into large, thick blocks that could support their own weight when stacked, either dry or with mortar. However, with the advent of lightweight structural framing systems and curtain wall construction in the 19th century, the weight and durability that once made stone appealing also made it increasingly difficult to fabricate and handle economically, hindering its adaptation to modern construction methods.

This changed through General Electric's development of high-quality synthetic diamonds and carbide abrasives for industrial applications and in making thin stone slates for use as curtain wall elements. The General Electric development occurred in several decades spanning over 1950's to 1980's, making the fabrication of thin stone slates, ranging from 2 to 4 inches (51 to 102 millimeters) in thickness, more cost-effective for façade applications. Later developments introduced adhesive technologies for stone slate attachment.

Today, stone can be fabricated in thicknesses as low as three sixteenth to one quarter of an inch (5 to 6 millimeters), thanks to continuous innovation. Another significant breakthrough came in the 1950s when Henry M. Rothberg, a chemical engineer and founder of LATICRETE International, invented a method and product that enabled the direct adhesive attachment of ceramic tile, stone, masonry veneer, and thin brick to exterior facades. This innovation revolutionized both the ceramic tile and stone industries, making their application on building façades more practical and economical once again. An example of a modern application of a stone façade is shown in Figure 4.3.

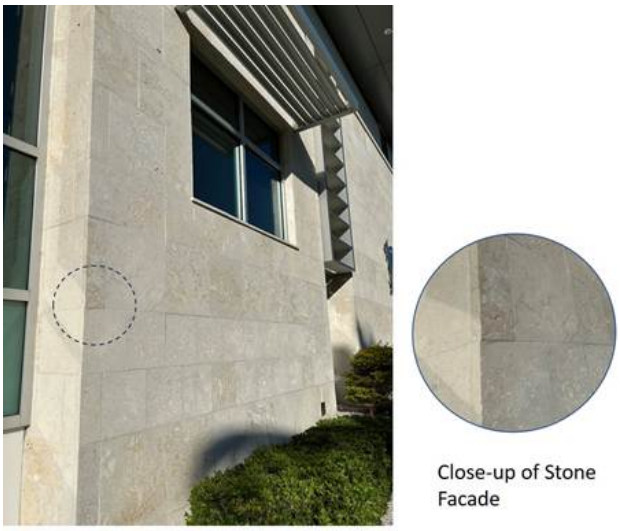


Figure 4.3 An example of a stone façade (from personal collection of the lead author)

(This figure shows a photograph of a stone façade with a close-up detail given to the right of the photo.)

4.2.1: Popular stones in façade applications

4.2.2: Stone characteristics and properties

4.2.3: Distress conditions

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4.2.1: Popular stones in façade applications

Stones that are popular in building façade applications can be divided into the following major groups.

- Sedimentary stones
- Igneous stones
- Metamorphic stones

Sedimentary stones – This group includes limestone (including travertine), sandstone, brownstone, and shale. These stones form from the deposition of sediment in riverbeds, lake beds, and other bodies of water. The sediments originate from the erosion and breakdown of pre-existing rocks, minerals, and organic matter, which become compacted and cemented together through natural geological processes. Sedimentary stones are characterized by distinct bedding planes, representing individual layers of material, and by variations in grain size. Limestones are particularly favored for façade applications and are often used in conjunction with other masonry materials especially in load-bearing wall construction.

Igneous stones – Granite and schist are among the most well-known stones in this category, formed through volcanic activity when molten magma cools and solidifies. Igneous rocks are typically rich in quartz, the crystalline form of silica, and are classified based on their silica content. The surface finishes applied to igneous stone panels, particularly granite, can significantly influence both the architectural appearance and the structural performance of a system. Various finishing techniques, such as flame finishing, bush hammering, and other abrasive treatments, can induce microcracks in the surface of the stone. Microscopic analysis has revealed cracks penetrating to depths of 1-8th to quarter of an inch (3 to 6 millimeters), depending on the specific treatment used. Given that stone panels are often only 1.25 inches (32 millimeters) thick, the reduction of effective thickness by a quarter of an inch (6 millimeters) can substantially affect the stone's handling during installation, as well as its long-term performance when exposed to weathering, particularly freeze/thaw cycles.

Metamorphic stones – This group includes marble and slate, materials long valued in architectural façade applications, both as veneers and ornamental elements. Metamorphic stones form when sedimentary or igneous rocks are subjected to extreme heat and pressure over millions of years, leading to the recrystallization of their mineral structure. Two primary types of metamorphism drive these transformations. The first, thermal metamorphism, occurs when rock is exposed to prolonged heat in a confined environment, as seen in the conversion of limestone to marble. The second, regional metamorphism, is driven by tectonic forces, such as those involved in mountain formation, where rocks experience sustained pressure. In this process, the recrystallization leads to the alignment of new mineral grains in response to the directional forces. Slate is a common example of rock produced through this regional metamorphism.

Metamorphic stones are classified based on the type of rock they originated from before metamorphism. Due to their interlocking grain structure, these stones are prone to a phenomenon known as hysteresis (Loughran, 2007), which reflects their nonlinear and inelastic behavior under cyclic thermal loading and unloading. This thermal cycling occurs when the exposed surface of a panel undergoes expansion due to heat, while the unexposed side remains cooler, leading to differential expansion. Upon cooling, the grains do not return to their original arrangement, causing permanent elongation of the heated surface. Repeated cycles of heating and cooling can lead to the permanent bowing of the stone panels (Tschegg and Krec, 2009).

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4.2.2: Stone characteristics and properties

Being a natural material, the physical and aesthetic properties of stone can vary significantly, even within a single quarry. The inherent variability of stone is influenced by several factors:

- Natural planes of weakness, such as cleavage planes, bedding planes, and rifts, are discontinuities present within the stone's matrix.
- The physical characteristics of stone are further affected by moisture conditions, with properties varying between wet and dry states.
- Stone is anisotropic, meaning its strength varies depending on the direction of the applied load.
- Stone is also heterogeneous, contributing to its overall variability in behavior and performance.

Each geological category and formation process imbues stone with distinct physical properties. Over the last several decades, stone fabricators and distributors have developed standardized testing procedures and established minimum property requirements. Historically, however, stone was used in large, compact shapes primarily subjected to compressive forces. In these applications, the massive nature of the structure absorbed lateral loads, which were resisted by the geometry of the building rather than individual components. The transition from load-bearing stone masonry systems to modern individually supported stone panels was neither smooth nor uniform. Early cladding systems were based on empirical methods rather than precise knowledge of material properties.

Key recognized properties of stone include compressive, flexural, shear, and tensile strengths, as well as density, abrasion resistance, thermal expansion coefficient, and modulus of elasticity. However, there are additional, often overlooked properties that can significantly impact the long-term performance of stone cladding systems. These include permanent volumetric changes (hysteresis), freeze-thaw durability, and chemical and thermal weathering. Stone finish and permeability also influence its durability. These factors affect the structural integrity of the stone, with performance varying by stone type. Of particular concern to architects is the chemical and thermal weathering of stone when used in façade cladding. For example, certain types of stone can chemically interact with atmospheric carbon dioxide (CO_2) and water, leading to material dissolution at anchor points, which may result in cracking or, in extreme cases, detachment of cladding units.

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4.2.3: Distress conditions

Stone distress conditions can manifest in various forms, including discoloration, dislocation of façade units, cracking, corrosion or loss of metal anchor and rebar connections, spalling, sealant deterioration, and chipped or fractured pieces. While these issues are often identifiable through close visual inspection, further investigation may sometimes be required. In such cases, stone cladding units and blocks may need to be removed to assess underlying conditions, including the state of the metal anchors and corroded rebar connections, see, for example, Hoigard and Mulholland (2003).

When deformation of stone cladding – particularly in marble – is suspected, additional tests on removed samples may be necessary to determine the severity of the damage. These tests help evaluate whether the deformation poses a significant risk to the overall structural integrity of the façade.

Table 4.2 summarizes several common distress conditions in stone façade panels, the damage appearance, causes and potential lines of action to address each distress condition.

Table 4.2 Common distress conditions in stone façade panels – causes and lines of actions

Distress Condition	Damage Characteristics	Causes	
Bulging & displacement	Dislocated panels, open joints between panels, rust stains appearing	Failure or loosening of metal anchors; corroded metal anchors; deflection or failure of support framing system; ice buildup between backup masonry and panels; stone material chipped away or chemically dissolved at metal anchor connections; hysteresis cycles	Minor displacement repair weep holes Major displacement understand underlyi (f needed), (3) repla panels from extra lo hole, if needed
Peeling & delamination	Separation of thin surface layers; peeling; patches of stone missing on surface	Long term effect of rain water, along with dissolved CO_2; icing; natural bedding; bird dropping; splashed pavement deicing (in panels close to ground)	Minor damage: clea compounds Major damage: repl
Water infiltration	Separated panels, open joints between panels, rust stains appearing; cracks in panels	Lack of proper water management system (flashing and weep holes); deteriorated sealant in joints	Minor damage: (1) a repair cracks with s keep panel under ob Major repair: replac system and new me
Metal anchor failure	Loose panels, separated panels; rust stains; missing panels; large open cracks in panels	Faulty design, poor selection of kind of stone; Long term effect of rain water, along with dissolved CO2; ice buildup	(1) Replace panels (Install new metal ar

Chipped & fractured pieces	Missing and chipped away pieces (especially at corners)	Mainly because of composition of stone material; damage during installation	Minor damage: (1) g apply patching and Major damage: repl
Discoloration	Panels different in color than other parts of façades; dark color appearing on panels either spotty or widespread; patchy green color	Effect of pollutants in the air mixed with rain water; damage from bird dropping; vegetation growth; algae growth	(1) Wash and clean a approved non-abras necessary and per r

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4.3: STUCCO

Stucco, sometimes referred to as render, in the context of facades, is a durable, plaster-like exterior finish composed primarily of Portland cement, sand, lime, and water. It is typically applied in multiple layers over a substrate such as metal lath, wire mesh, or masonry to create a solid, weather-resistant surface.

From a technical and engineering standpoint, stucco serves both as an aesthetic and functional component. It provides a hard, breathable finish that allows moisture to evaporate while protecting underlying structural elements from environmental factors like water infiltration, temperature fluctuations, and UV radiation. Stucco is also known for its versatility, allowing it to be textured or colored to meet various architectural requirements.

Stucco facades are constructed with several layers (see Figure 4.4). These are

Substrate preparation and finish – In nearly all applications, the façade surface must be properly prepared. Often a water-resistant barrier and lath is installed. For such substrate material as masonry and concrete, the stucco can directly be applied. In this kind of application, the stucco will need be designed as a barrier wall system against moisture penetration. And as such, usually a waterproof coating is also applied over stucco.

When wood or metal panels are used as the substrate in façades, a layer of building wrap and metal lath is required to provide a base for the stucco.

Scratch coat – Making the first layer of stucco, the scratch coat is a mortar-like material made up of cement, sand, and lime. When applying, the coat is scored or scratched (hence the name) to provide a better attachment of the next layer. The scratch coat is usually about 1/4 to 3/8 of an inch (about 6 to 10 millimeters) thick.

Brown coat – This coat is the second layer in a stucco finish. It uses a similar mixture to that used in the scratch coat. However, the mixed design often contains a higher proportion of sand. The thickness is generally about the same as that in the scratch coat or even slightly thicker, up to about 1/2 of an inch (13 millimeters). This layer provides a rather smooth surface for the finish coat.

Finish coat – The final coat provides the finish look of the stucco. The material consists of fine sand, cement, and often pigments for a desired color. This layer is only about 1/8 of an inch (3 millimeters) thick. Proper material design for this layer, in terms of sand contents and aggregation, is important in maintaining a durable material with less vulnerability to crack development and delamination.

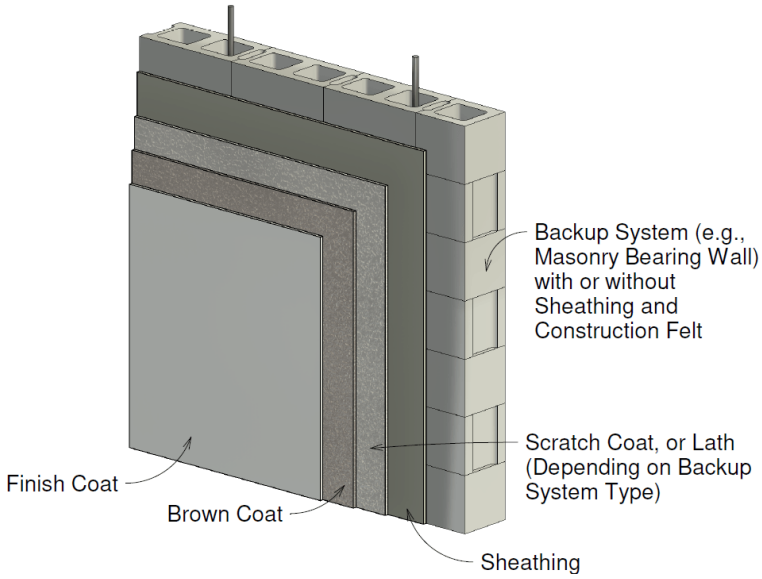


Figure 4.4 Stucco layers

(This figure shows a detail of several layers making up stuccos. The layer on the surface of the structure is the scratch coat, the next one is the brown coat and the one exposing to outside is the finish coat.)

4.3.1: Construction process

4.3.2: Distress Conditions

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4.3.1: Construction process

This process is especially important to be followed when upon façade inspection, it is decided to reconstruct a portion or the entire façade. In situations when lath is required, the lath is applied along with a building wrap and secured via nails or screws. When scratch coat is needed, it is applied in lines or scratched as explained earlier. The curing of this layer may require several days depending on the weather conditions. The brown coat is then applied allowing again several days for curing. The final finish coat is then applied with an ample time allowed to cure completely.

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4.3.2: Distress Conditions

In most applications, a major distress condition in stucco is crack formation. In some situations, these cracks are the grazing type, which at times are more like patchy hairline cracks that are only detectable with close examinations. More severe cracks run over longer distances and are often visible afar. Delamination and separation of stucco are also common problems, which are generally detectable afar. In general, most distress conditions in stucco stem from improper mixture design for various layers, poor workmanship in applying stucco layers, inadequate thickness, and issues with lath installation and encapsulation, as recommended by ASTM C926 - Standard Specification for Application of Portland Cement-Based Plaster (see, for example, Cope and Horst, 2014). Table 4.3 provides an overview of stucco distress conditions, the damage appearance, causes, and the line of actions that will be necessary to address each condition.

An example of stucco damage in a CMU wall in the form of delamination and separation is shown in Figure 4.5.

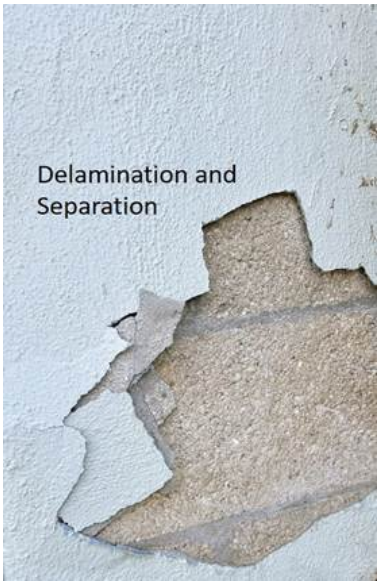


Figure 4.5 Delamination and separation on a stucco façade (from personal collection of the lead author)

(This figure is a photograph of a portion of the finish coat separated through delamination.)

Table 4.3 Common distress conditions in façades made up of stucco – causes and lines of actions

Distress Condition	Damage Characteristics	Causes
Stucco cracks	Fine lines of crazing cracks; lines of opening in façade; portions of stucco appeared being separated	Distress conditions resulting in tension in stucco; poor workmanship and curing of substrate and finish mixtures, incorrect stucco mixture and gradation; weathering; external forces; structural movement; corroded lath; lack of expansion joints

Peeling & delamination	Separation of thin surface layers; peeling; patches of stucco missing on surface	Poor workmanship and curing, incorrect stucco mix and gradation; weathering
Separation	Portions of stucco missing or appeared bulging, lath visible	Poor workmanship and curing, incorrect stucco mix and gradation; weathering, external forces, structural movement; corroded lath; lack of expansion joints
Lath damage	Spotty cracks; rust stains; portions of stucco missing or appeared bulging, lath visible	Faulty design, lack of lath encapsulation in mixture; improper finish mixture; poor installation and lack of proper attachment to the backup wall or structural system; improper lapping of lath
Discoloration and efflorescence	Panels different in color than other parts of façades; dark color appearing on panels either spotty or widespread; patchy green color; lines of white powdery appearance	Effect of pollutants in the air mixed with rain water; damage from bird dropping; vegetation growth; algae growth; inappropriate mixture of finish material; presence of salt compounds in the mixture

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4.4: SYNTHETIC STUCCO

Synthetic Stucco was developed by BASF Chemical Company in the late 1950's and was first used in Europe as an efficient way of restoration of damaged properties. Currently, this system is known as the Exterior Insulated Finish System (EIFS). Basically, it is a wall façade system that also has an insulation layer, which is applied on the exterior of the building walls along with a finish (hence the name). The finish is comprised of polymeric bonded aggregate and cement reinforced with a glass mesh. Although it appears similar to stucco, the two perform very differently (Lstiburek, 2007). Over the years, the composition of EIFS has changed. In most current designs, EIFS cladding is made up of six layers (see Figure 4.6). These are

- Water-resistive barrier (WRB)
- Drainage plane
- Insulation board (usually expanded polystyrene), about 2 inches (51 millimeters) thick
- Glass-fiber reinforcing mesh
- Water-resistant base coat
- Finish coat

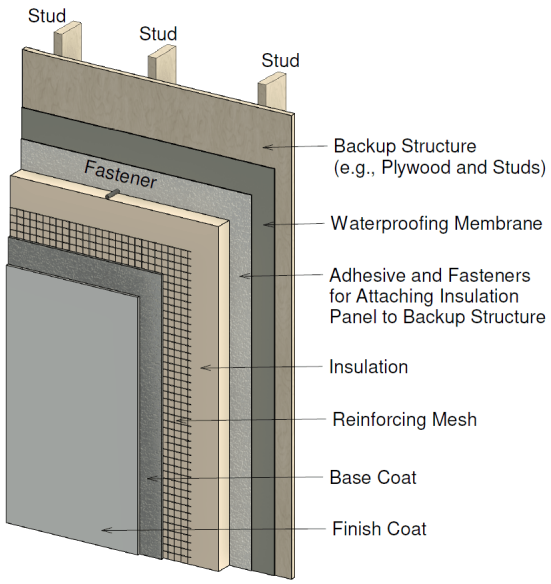


Figure 4.6 Layers in external insulation and finish system

(This figure shows various layers making up the external insulation and finish system. The layer on the structure is a waterproofing membrane, next one is the adhesive, the next one is the insulation, then a reinforcing mesh, a base coat and the finish coat are shown.)

As the finish is often mistaken by stucco, a quick determination on whether a finish is EIFS or stucco is simply by tapping the façade in several locations. A hollow sound is an indication of EIFS while real stucco will have a more solid sound. However, the determination of the types and kinds of layers use, the identification of any distress, and recommendation for implementing a proper line of action will generally require hiring the services of an EIFS expert.

4.4.1: Distress conditions

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4.4.1: Distress conditions

Distress conditions causing damage to EIFS are primarily related to moisture and leakage. This situation stems from a variety of factors including weather condition, frequency of rain, damage to the finish and insulation layer, and improper sealing around openings used for windows.

Kottra and Summers (2017) lists the following deficiencies that contribute to the leakage situation in a case study report.

- Failure of primary and/or secondary sealant at joints between EIFS panels
- Failure of bond between EIFS finish coat and base coat
- Extension of EIFS base and finish coats onto supporting studs, thereby facilitating a path for moisture migration through the thickness of the wall panels into the inside the building
- Lack of back-wrapping of the base coat behind the EIFS panels

In areas with humid climates, the limited drying potential provides inadequate drying for EIFS, especially when moisture sensitive components are used without a provision for drainage. Since, inward drying is essentially eliminated by the installation of interior vapor barriers or impermeable interior finishes such as vinyl wall coverings, the moisture build-up will result in mold and other damage scenarios such as decay and corrosion.

Other damage scenarios in EIFS claddings include cracking, impact damage, systems detachment, and surface degradation (see, for example, Lampo and Trovillion, 1990). The causes for these damage scenarios vary from one application to another. In addition to weathering and human-induced damage through impacts, poor workmanship and installation errors are also among the causes. Cracking and separation can occur as a results of the building movement due to temperature variations, concrete deformation, and steel structure deflection. Table 4.4 provides an overview of EIFS distress conditions, the damage appearance, causes, and the line of actions that will be necessary to address each condition.

Table 4.4 Common distress conditions in façades made up of EIFS – causes and lines of actions

Distress Condition	Damage Characteristics	Causes
Interior moisture build-up	Persistent wetness inside, mold build-up in interior walls; wood decay, metal rust	Humid climates; limited drying potential; excessive exposure to rain; lack of provision for drainage; installation of interior vapor barriers or impermeable interior finishes such as vinyl wall coverings
Cracks and detachment	Extended cracks; localized finish wrinkling or bulging; loss of sealant around windows	Poor workmanship; weathering; structural movement; frequent temperature variations; improper insulation attachment, inadequate substrate layers
Impact damage	Surface finish pieces missing or dented or bruised, spotty detachments	Low impact-resistance material; hits by external obje (e.g., landscaping machinery); poor or damage mesh and substrate materials

Surface degradation and discoloration	Areas appeared fading; spotty surface finish loss; spotty rust stains on surface; pieces of metal and mesh visible	Weathering; improper finish; improper installation of mesh and substrate materials; improper fastener installation

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4.6: Exercises

E4.1 What constitutes a distress in terra-cotta units called crazing?

a) Spotty cracks with random patterns on the glaze.

b) Lack of glaze.

c) Major cracks on the glaze extended into materials beneath.

E4.2 How does one suspect corrosion of metal anchors supporting terra-cotta units?

a) Terra-cotta units appear dislocated and loose.

b) Terra-cotta units have cracks.

c) Terra-cotta units have spotty damage areas on surface such as glaze spalling.

d) All of the above.

E4.3 The selection of a proper replacement for damaged terra-cotta units is an important consideration in deciding on façade reconstruction because

a) It is absolutely impossible to find an exact replica of damaged units.

b) There are presently no manufacturers left that can make a like unit.

c) If not properly selected, replacement units may change color or deteriorate affecting the façade architectural aesthetic.

E4.4 Crazing in terra-cotta units, identified by small random cracks in the glaze, generally requires replacement when:

a) Cracks extend into the porous tile body beneath the glaze.

b) Cracks are visible on the surface.

c) The unit is exposed to moisture.

d) The surface has discolored.

E4.5 Which of the following factors influences the modern application of stone as façade claddings?

a) Availability of large slates of different color stones.

b) Advent of new method in cutting stone.

c) Declining cost.

d) Availability of ore modern mining equipment.

E4.6 Materials such as limestone, travertine, sandstone, brownstone, and shale belong to

a) Metamorphic stones group.

b) Sedimentary stone group.

c) Both Sedimentary and metamorphic stone groups.

d) A group different from metamorphic and sedimentary.

E4.7 Among the following factors related to the location of a building, which one is considered most effective in causing damage to EIFS facades?

a) Cold regions.

b) Dry hot regions.

c) Wet and humid regions.

E4.8 In identifying a potential damage to the underlying lath in a façade made up of stucco, which of the following statements is most nearly accurate?

a) There are always signs of rust.

b) There are always signs of spotty cracks.

c) There are always signs of stucco bulging.

d) There are one or more of signs indicating rust, bulging or spotty cracks.

E4.9 What is a primary cause of lath damage in stucco façades?

a) Incorrect mixture of finish material.

b) Improper lapping of the lath and lack of proper attachment to the structural system.

c) Frequent exposure to pollutants and bird droppings.

d) Inadequate expansion joints.

E4.10 Among the following, what is a more prevalent cause of bulging and displacement in stone façade panels?

a) Ice buildup between backup masonry and panels.

b) Poor sealant in joints.

c) Bird droppings.

d) Vegetation growth.

E4.11 Discoloration on stone façade panels, such as dark or patchy green spots, is often contributed to:

a) Composition of stone material.

b) Poor water management system.

c) Pollutants mixed with rainwater or algae growth.

d) Deflection of support framing system.

E4.12 What is the most appropriate course of action when addressing extensive cracking in a stucco façade?

a) Treat the cracks as superficial and seal them with caulk.

b) Clean the surface and apply a weather-resistant paint without repairing the cracks.

c) Rebuild a significant portion or the entire stucco façade, incorporating expansion joints where necessary.

d) Repair the cracks using a rigid mortar-based filler.

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5: BUILDING ENVELOPE INSPECTION PROCESS, AND ADMINISTRATION AND MANAGEMENT MATTERS

Chapter Synopsis

This chapter covers the basic elements of a successful building envelope inspection process, starting from the planning stage to the completion. An overview of the administration and management matters, as related to the operation of a building façade inspection business, is covered in this chapter. Furthermore, building envelope inspection ordinances in several major cities in the United States and Canada are also reviewed and summarized.

5.1: Introductory Remarks

5.2: Planning for the Visual Inspection

5.3: Initial Engineering Analysis and Evaluation

5.4: Conducting the Inspection

5.5: Preparation of Report and Forms

5.6: Administrative and Management Matters

5.7: Chapter References

5.8: Exercises

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5.1: Introductory Remarks

The building envelope inspection process is considered an engineering activity. As in any other project, the entire process involves the following three major steps.

- Planning
- Engineering analysis and evaluation
- Execution (i.e., the actual conduct of inspection)

As part of this process, various kinds of preliminary investigations, data gathering, and reviews are essential for a cost effective and successful inspection activity. Preliminary investigations also include a complete review of rules and regulations governing building envelope inspections as mandated by the local jurisdiction or municipality. Since many such regulations, especially in large urban areas, require that a licensed structural engineer, registered professional engineer, or architect oversee the inspection process and related activities, it is important to know the type of registrations, training, or certifications that are needed by the local authority before starting the inspection process.

As in any type of professional activities, the administration and management of the process are conducted within practices that are acceptable by the profession. In this regard, the professional of the record is the managing director in the inspection process. Whether the professional acts as a sole proprietor, owner of a small company or employee of a larger corporation, there are certain rules that they will need to follow in regard to the management, relation with others, and everyday operation of the organization.

In this chapter, the basic elements of a successful building envelope inspection process, embodied in the three aforementioned steps, are presented. It is emphasized that subjects covered are intended as general information as additional rules may apply to specific areas as required by the local authorities. Furthermore, depending on the special characteristics of a building, certain actions may be required that are unique to the building and may call for hiring services of specialists as consultants.

This chapter also presents a brief overview of the administration and management matters, as they relate to the operation of the business of building façade inspection.

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5.2: Planning for the Visual Inspection

The building envelope process begins with strategic preparation, which involves developing a comprehensive and detailed project document. Several preliminary activities should be completed before this document is finalized, including:

- Collecting relevant data.
- Preparing a schedule of activities with estimated hours for each task.
- Compiling a preliminary list of personnel assignments.
- Reviewing applicable safety requirements.
- Preparing an inventory of required equipment.
- Coordinating with subcontractors and consultants to ensure timely delivery of equipment and services.
- Arranging initial meetings with all personnel to provide briefings on task assignments, safety protocols, reporting requirements, documentation, and overall inspection management.

The project document consolidates all essential information, including inspection procedures, building characteristics, safety protocols, and the full schedule of planned tasks and activities.

Collecting relevant data – This is an important part of the planning phase of the inspection process. It involves obtaining all pertinent documents available for the building including, but not limited to, those summarized in the following list.

- Construction type, composition of the façades on all sides, structural system and type, roof systems, and presence of parapet walls and their types.
- Occupancy type, e.g., residential, commercial, or mixed use.
- Building dimensions, boundaries, types of access roadway (major street, local throughfare, and alley), and information on the neighboring properties.
- Building age and history of any major remodeling and changes in occupancy.
- Information on all ancillary structures such as external fire-scape stairs, canopies, balconies, porches, water tanks, flagpoles, and chimneys that use the facades and roof as support.
- Records of any safety violations or citations by the local municipality.
- Copies of previous inspection reports and records of major repairs to the façades and roof.
- Copies of architectural, structural, mechanical, plumbing, and electric design drawings.
- Information on the building ownership (address and contact numbers).
- All documents and guidelines published, and forms that are required by the local municipality.

Initial visits to the site are needed as part of the planning to gather any additional information and features that are unique to the building. Furthermore, it is important to learn about the overall accessibility of the building for façade inspection, especially when scaffolds are needed to be installed.

Schedule of activities – This schedule is needed in arriving at a preliminary estimate for the required manpower and arranging for the coordination between various activities. It is of course expected that this schedule will be revised as the work progresses to accommodate any new findings at the site and the need for additional work and/or change orders requested by the subcontractors.

In situations when the inspection requires close examination aboard a moveable scaffold, the schedule of activities will need to clearly identify the sequence and time of removal and re-installation of the equipment in a manner that will minimize the downtime. This is of course done in consultation with the scaffold service subcontractor. Stationary scaffolds require longer times for initial installation. This needs to be included in the schedule of activities. Furthermore, the schedule of activities must include the hours of operation and a plan for an efficient coverage of various portions of the façades as certain pieces of equipment will need to be moved from one part to another, which requires proper coordinations among workers.

Personnel assignment – A table summarizing the task assignment of personnel involved in the inspection process is important as a way of manpower distribution and management and proper allocation of resources. This distribution of manpower and tasks is preliminary; and as the project makes progress, adjustments may become necessary to accommodate changes, delays, and treatment of newly discovered areas that need attention.

Safety – Safety is a fundamental priority in any façade inspection project. Accordingly, the planning phase must include a

comprehensive compilation and clear documentation of all applicable safety rules and regulations. These measures are essential to safeguard not only the inspection personnel but also building occupants, bystanders, and pedestrians.

In circumstances where local regulations mandate that façade inspections be conducted from scaffolding - particularly moveable scaffolds - inspection personnel may be required to complete specialized safety training and obtain appropriate certifications. Such training ensures compliance with the Occupational Safety and Health Administration (OSHA, 2002) requirements, as well as any additional state or municipal regulations. To prevent delays once the project begins, these requirements must be clearly identified and documented during the planning phase. By doing so, all inspection personnel can obtain the necessary credentials in advance, ensuring that the project proceeds safely, efficiently, and in full compliance with applicable regulations.

When moveable scaffolds are employed, and particularly before any rope descent system (RDS) is used for worker access and safety, each anchorage point for the RDS must be verified as capable of sustaining a minimum load of 5,000 pounds (22.2 kilonewtons) applied in any direction. OSHA regulations (2017) require that this load-carrying capacity be documented through inspection and certification by a qualified person on an annual basis, with a comprehensive recertification at least once every ten years. For this reason, it is essential to fully understand the type, configuration, and strength of the structural members located on the roof that serve as supports for the anchorage system. Without such verification, the anchorage system may present unacceptable safety risks to workers.

Because the recommended load may be applied in multiple directions, the beams or structural members supporting the anchors may experience torsional stresses in addition to biaxial bending moments. Consequently, a thorough investigation of the structural condition and load-carrying capacity of these members is required to ensure they can safely resist both the OSHA-prescribed anchorage forces and the existing roof loads. In many cases, the expertise of a licensed structural engineer will be necessary to assess adequacy, identify any deficiencies, and design appropriate retrofits or reinforcements. This step is critical not only for regulatory compliance but also for ensuring worker safety and maintaining the structural integrity of the roof system.

Equipment needed – Although each façade inspection project has its own unique characteristics, there are certain categories of equipment that are universally required across all inspections. To ensure efficiency and safety, a comprehensive equipment inventory should be developed during the planning phase and carefully reviewed with all inspection personnel prior to mobilization. This list should not only specify the essential tools and devices but also highlight those items that demand periodic verification for functionality, calibration, and readiness. Particular attention should be given to electronic equipment - such as laser measuring devices, crack gauges, drones, mobile phones, tablets, and digital cameras - since their accuracy and reliability often depend on proper calibration and adequate battery power or charging capacity. Establishing a routine protocol for checking, charging, and documenting the operational status of these devices is critical to minimizing disruptions in the field and ensuring that data collected during inspections are accurate and dependable.

Table 5.1 summarizes a list of most common pieces of equipment and their purposes. This list is intended as a general guide. It is noted that certain additional pieces of equipment may be necessary depending on any unique situation that may be present.

Table 5.1 List of common pieces of equipment needed in façade inspections

Item	Application and purpose
Backpack	Carrying pieces of equipment, water, and snack
Binoculars	For afar observation of façade conditions
Camera	Taking photos of the overall façade as well as areas of distress observed
Clipboard	For taking notes, preparing sketches, etc.

Comparator	This a credit-card size ruler used for measuring crack width
Dental examination mirror	For observing the condition behind a barrier, especially when cracks are extended behind a masonry element (for example, in a coping stone on a parapet wall)
Documents	Credentials and certificates per requirements by local authorities
Façade sketches	These sketches are handy for marking up locations of distress conditions on them
Field notebook	This is a pocket-size notebook, similar to those used by land surveyors - it is needed for writing observation notes, drawing sketches, and writing locations of observed distress conditions
Flashlight	Observing areas behind masonry blocks where light is needed
Hard hat	Personal protection
Markers and pens	For taking notes, marking on façade elements, if necessary
Magnifying glass	Magnifying glasses are needed when small cracks are hard to be distinguished from other features
Measurement tape and ruler	Measuring sizes of masonry elements and distances
Mobile phone	Communication and other applications available in smart phones
Piano wire	For measuring crack width
Spray paint (the type that can easily be cleaned if needed) and thick-tip markers	For marking on façade elements, if necessary
Video camera with a lens mounted at an extended flexible line	This device comes handy to observe hard-to-reach areas and also behind barriers

Coordination of activities with subcontractors and consultants – Depending on the type of building, its façades, and the extent of the inspection contract, various expertise and skill sets may be required. And as such, services of subcontractors and consultants are sought in the project. The preliminary planning needs to include the schedule of activities by these services, the extent of their involvement, and reporting requirements. For on-site services such as scaffolding, removal of masonry blocks,

sampling, and non-destructive tests, the plan must also include the extent of safety and operation requirements as mandated by the local codes and regulations.

Initial meetings – The planning process begins with a series of initial meetings to introduce personnel, subcontractors, and consultants to the project and provide an overview of all required tasks. A planning document is then shared with all participants. To ensure its accuracy and completeness, input and feedback from all involved parties are gathered, allowing for necessary revisions, additions, and adjustments to the planning document.

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5.3: Initial Engineering Analysis and Evaluation

Once previous inspection reports and all available design and construction drawings have been obtained, it is often necessary to carry out a series of preliminary engineering analyses and evaluations. The purpose of these assessments is to identify additional tasks or preparatory activities required to ensure the safe and effective execution of the façade inspection. Such activities typically arise from the unique architectural, structural, and operational features of each building. At a minimum, the evaluations should consider the following key factors related to façade inspection:

- Structural adequacy of the façade surfaces or selected portions thereof, specifically with respect to their ability to safely support temporary loads from scaffolding, platforms, and other inspection equipment.
- Potential obstacles and physical impediments (e.g., overhangs, setbacks, signage, vegetation, or building appurtenances) that may complicate access or movement of inspection personnel and equipment.
- Accessibility conditions on all building elevations, including the need for easements or permissions to access façades from adjacent properties or public rights-of-way.
- Roof condition and load-carrying capacity, particularly in areas designated for installation of anchorage points, rope descent systems, or other fall protection equipment required to safeguard personnel working on suspended scaffolds.

As expected, the scope and complexity of these analyses vary from one building to another. Therefore, in addition to reviewing historical reports and design documentation, it is critical that the inspection team conduct one or more site reconnaissance visits. These visits allow for verification of existing conditions, identification of unanticipated challenges, and documentation of any features that may require special consideration in the planning and execution of the inspection.

Table 4.2 provides a summary of the most common types of initial engineering analyses and specific sources for obtaining data that is needed for conducting such analyses.

Table 5.2 Initial engineering analyses and evaluations

Kind of analysis and evaluation	Purpose	Source
Envelope system construction	Understanding the type of façades; backup wall system; support system used for brick; stone; CMU or terra cotta blocks	Architectural drawings
Structural system	Understanding the load-bearing system; type of framing used; type of construction (steel, timber; concrete or masonry); understanding the type of foundation	Structural drawings
Stability and strength of façades	Determining façade's condition to ensure safety of personnel and pedestrians	Previous inspection report; s the site and evaluating façad
Strength of roof beams	Investigating whether strength is adequate to support anchors for moveable scaffolds and fall protection ropes	Structural drawings
Roof condition	Identifying easy-access areas; locating roof-installed equipment (such as air conditioning) that may limit installation of scaffold frames and weights	Mechanical drawings, visits t

Accessibility of façades on all sides	Planning for performing inspection, securing permission from adjacent building owners; finding alternate way of accessing areas on façades	Architectural drawings, previ to the site and evaluating faç
Stability of parapet walls	Determining wall's condition to ensure safety of personnel and pedestrians	Visits to the site and evaluati
Elements and features on façades	Identifying locations of ornamental pieces; balconies; flag poles; fire escapes; and other features that are obstacles to façade inspection	Architectural drawings; previ visits to the site
Windows, doors, and other openings	Determining the condition of window and door openings	Previous inspection reports; repair and replacement
Specialized analyses	Understanding any other type of special conditions that may require the use of nondestructive test methods and long-term monitoring including investigating asbestos	Previous inspection reports; consultants on condition ass asbestos issues; strengthenir etc.

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5.4: Conducting the Inspection

Major city ordinances generally require supervision by a licensed architect, structural engineer, or a registered professional engineer when conducting the inspection (see Table 5.3). Depending on the type of building, occupancy, height, and location, the inspection may be cursory or detailed. The former is sometimes referred to as a routine inspection or examination and is usually conducted with the aid of binoculars from afar and more frequently than a "detailed inspection." The detailed inspection is also called the critical inspection or examination. Based on the types of façade coverage, the extent of inspection and the type of equipment needed for each type are different. Detailed or critical inspections are conducted in longer intervals and usually require close examination of a sufficient area of each façade aboard a scaffold. It is important to note that the documents pertinent to inspection and examination of façades, for the municipality in which the building is located, will need to be carefully reviewed for full compliance with the local rules and regulations.

Cursory or routine inspection – This activity is required to be carried out on a more frequent basis and serves as a means of verifying that the overall condition of the façades remains stable and that no significant deterioration or distress has developed since the last inspection. In most jurisdictions, this type of assessment is mandated on an annual basis or at the mid-point between comprehensive "critical" inspections. For example, the City of Chicago Façade Inspection Ordinance specifies this requirement and classifies it as an "on-going inspection" (see Table 5.3).

Because this type of inspection is essentially a follow-up activity, the primary emphasis is placed on those areas previously identified as requiring surveillance or close observation. In practice, scaffolding or rope descent systems are not typically employed. Instead, the evaluation is performed from the ground or nearby vantage points using binoculars or other optical aids. Following a focused examination of the designated target areas, a general review of other portions of the façade is conducted to confirm that no new or unanticipated distress conditions have emerged.

As noted in Table 5.1, annotated sketches of the façades are particularly useful for recording changes in previously observed conditions such as cracks, loss of mortar, displaced or loose masonry units, and deterioration of architectural elements such as terra-cotta. These sketches should be supplemented with an adequate number of high-resolution photographs to ensure that the current façade condition is thoroughly documented. For each distress condition, field notes must be prepared describing the nature of the defect, its precise location, and its dimensions. Where appropriate, detailed sketches of the specific defect may be prepared to supplement the photographic record. Together, the photographs, field notes, and sketches serve as essential inputs for preparation of the final inspection report.

It is strongly recommended that the documentation process begin immediately upon returning from the field, while the observations remain fresh in the inspector's mind. Drafting the relevant sections of the report promptly ensures that critical details are not overlooked and that the inspection record remains both accurate and comprehensive.

Detailed or critical inspection – This activity is generally required to be performed in longer intervals – for example, 5 years by most major cities (see Table 5.3). Generally, a critical inspection implies a more thorough inspection that often requires close examination of façades aboard a scaffold. Again, sketches of the façades are helpful for marking any changes in the previously-observed and newly-appeared distress conditions such as cracks, missing mortars, displaced or loose bricks and CMUs, damaged terra-cottas, etc. These sketches are supplemented with photographs and detailed notes about the locations and types of distresses. Sketches prepared at the site, photographs, and notes are valuable information in preparing the final inspection report. As indicated, it is important to start writing the portion of the report for the inspection that is completed as soon as possible, while the information is still fresh in the inspector's mind.

Table 5.3 Building façade inspection ordinances in major cities in US and Canada (Mohammadi, 2021)

City	Requirement for inspection	Credentials needed for the pers inspection

Boston, Massachusetts	Buildings over 70 feet (21.3 meters) in height, or greater than 35,000 cubic feet (991 cubic meters) Interval: every 5 years; for unoccupied buildings, annually	Registered architect or engineer
Chicago, Illinois	Buildings 80 feet (24.4 meters) tall or taller Interval: between 4, 8 or 12 years, depending on the building category (ongoing inspection halfway into critical inspection intervals)	Licensed architect or structural engi
Cincinnati, Ohio	Buildings with 5 stories or more and a completed construction date of 15 years or more Interval: 8 or 12 years, according to an inspection schedule of and based on the building category	Registered architect or engineer
Cleveland, Ohio	Structures that are at least 30 years old and 5 stories or 75 feet (22.9 meters) tall Interval: every 5 years	Qualified inspector as defined by th housing Ordinances
Columbus, Ohio	Buildings more than 20 years old and located withing 10 feet (3 meters) of a public right-of-way, pedestrian walkway or plaza, excepting residential buildings up to 3 stories. Interval: every 5 years	Registered architect or engineer (fo in a designated downtown area)
Detroit, Michigan	Buildings 5 stories or greater in height Interval: every five years	Qualified (competent) person
Milwaukee, Wisconsin	Buildings of 5 or more stories and more than 15 years old Interval: varies between 5 and 12 years based on the building category	Registered architect or engineer
New York, New York	Buildings of 6 or more stories, including basements Interval: every 5 years	Registered architect or engineer (wi credentials to be considered as qua inspectors)

Philadelphia, Pennsylvania	Buildings of 6 or more stories or with any appurtenance over 60 feet tall Interval: every 5 years	Registered architect or engineer
Pittsburgh, Pennsylvania	All buildings and structures to determine the structural soundness of items covered in Sections 304.8, 304.9 and 304.11 of the property maintenance code Interval: all inspections made prior to the adoption of this code to continue on their previous schedule at five-year intervals	Licensed professional engineer or re
Quebec, Canada	All buildings of 5 or more stories Interval: every 5 years	Licensed architect or engineer
Saint Louis, Missouri	Buildings of 6 or more stories Interval: every 5 years Note: St. Louis uses the International Building Code (IBC)	Licensed design professional
San Francisco, California	Buildings of 5 or more stories Interval: periodically (every ten years, after the initial inspection of the building)	Licensed architect or engineer

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5.5: Preparation of Report and Forms

Most municipalities prescribe the format of the façade inspection report and specify the type of information that must be included. At a minimum, the report should clearly describe the condition of all building façades with respect to maintaining the safety of occupants, pedestrians, and the general public. In some jurisdictions, submission of standardized forms or official summary sheets is also required in addition to the detailed narrative report. Table 5.4 presents a general outline of the typical contents of an inspection report, independent of any mandated format. Nevertheless, the inspector must always review and comply with the requirements of the local authority having jurisdiction to ensure that the report conforms to the prescribed standards and formatting.

The inclusion of photographs, annotated illustrations, engineering drawings, and summary tables greatly enhances the clarity of the report and facilitates the communication of findings. Each photograph should be clearly labeled and cross-referenced to the specific observation being reported. Care should be taken, however, not to overwhelm the document with redundant images or repetitive information, as this can obscure the key findings and reduce the report's effectiveness.

Summary tables are especially useful for condensing information regarding the location and nature of observed distress conditions. A simple but systematic coordinate system can be developed for reference - for example, proceeding from bottom to top and left to right while identifying the location of defects in relation to floor levels and architectural or structural elements (such as window panels, columns, or spandrels). In some cases, color-coded markups of façade drawings are used to illustrate the distribution, extent, and dimensions of distress features such as cracks, areas of mortar loss, displaced or missing masonry units, and deteriorated terra cotta. These graphical tools allow both inspectors and stakeholders to quickly understand the severity and spread of façade deterioration.

Supplementary or supporting information, including calculations, extended photographic records, or specialized testing data, should be organized in an appendix to avoid cluttering the main body of the report. In certain municipalities, additional forms are required to accompany the inspection documentation. For example, the City of Chicago mandates submission of a specific "iron inspection" form, which summarizes the observed condition of metallic elements attached to the façade - such as chimneys, fire escapes, flagpoles, signage, piping, and ornamental metal features. This ensures that all building appendages with potential safety implications are evaluated in a consistent manner.

Table 5.4 Major sections of an inspection report

Subject	Contents and descriptions
Building information	Age; number of stories; dimensions; structure type; occupancy type; boundaries, bu (address); owner's information (address and contact number); and the date of prev inspections
Inspector's information	Name and contact information of the professional of the record and their credentia
Inspection information	Type of inspection (cursory, annual, ongoing, critical, etc.); duration of inspection; s inspection by date including meteorological data for each day,; areas covered duri along with drawings showing these areas; and references to specific articles in the ordinance and codes pertaining to the areas that have been inspected
Summary of findings by façade	Information on distress conditions; locations of cracks; dislocated and damaged fa and terra-cotta blocks; mortar damage and washout; and drawings showing the loc types of dislocations along with pertinent photographs

Recommendations	Description of condition (safe, unsafe, etc.); plan of actions (short-term versus long timetable
Forms and summaries	As required by the local rules and regulations
Supplementary materials	Photographs; drawings; sketches; list of documents and reports reviewed; and anal computations conducted as part of the façade inspection
Other observations	Summary of areas or conditions of building components that may impose safety c necessarily related to the scope of inspection, yet was observed as areas of conce be addressed by the owner)
Appendix	Any additional materials as supplement to the report and any special documents t required by the municipality to be included

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5.6: Administrative and Management Matters

When undertaking façade inspection or building envelope projects, several important considerations arise in the administration and management of the project, as well as in the overall business conduct of the professional or firm. Some issues are specific to sole proprietors or company owners, while others pertain to engineers employed by corporations, architectural firms, or those offering independent consulting services. Regardless of the employment setting, the professional must have a clear understanding of the following:

- Principles of effective project management.
- The nature of one's relationship with colleagues and employees.
- Protocols for communication with clients, contractors, consultants, and municipal authorities.
- The extent and limitations of professional liability.
- Proper procedures for record keeping, as well as financial and legal compliance.

Depending on project requirements, additional experts may be retained to support the inspection and reporting process. These may include other engineers, architects, accountants, or legal advisors. In such cases, it is essential that the roles and responsibilities of these specialists be clearly defined, documented, and understood by all parties - including the professional of record and their firm. Properly delineating responsibilities reduces the potential for disputes and ensures that accountability is maintained throughout the project.

This section provides an overview of administrative and managerial issues relevant both to the execution of inspection projects and to the operation of a professional practice. For clarity, the individual conducting or overseeing inspection activities will be referred to as the "professional."

- Key administrative and management topics relevant to façade inspection and building envelope rehabilitation include:
- Local municipal permit and licensing requirements.
- Liability insurance coverage appropriate to the scope of services.
- Contract law, including the definition of responsibilities and limits of liability.
- Business management practices, whether as a sole proprietor, partnership, or incorporated entity.
- Professional responsibilities to employees, including proper supervision, fair employment practices, and workplace safety.
- Consulting arrangements - whether hiring consultants to assist with specialized tasks or serving as a consultant to other firms.
- Effective communication protocols among all stakeholders, including owners, contractors, subcontractors, and consultants.
- Accurate and secure record-keeping practices.
- Regulatory requirements and procedures for establishing or registering a new business entity.

Permits – In nearly all jurisdictions, the repair, restoration, or rehabilitation of a building façade requires one or more permits issued by the local authority having jurisdiction, typically through the municipality's Department of Buildings or Department of Construction. While the building owner is ultimately responsible for securing these permits, in practice, the owner frequently delegates this task to the design professional of record (engineer or architect). Accordingly, it is essential that the professional be thoroughly familiar with the permitting process, the documentation required, and the associated responsibilities.

The permitting process generally requires the professional to:

- Prepare and submit permit applications along with supporting technical documents (drawings, specifications, engineering calculations, etc.).
- Respond to comments or requests for revision from municipal reviewers.
- Meet with municipal officials, such as plan reviewers, building engineers, or architects, to clarify project scope and compliance with codes.
- Resubmit revised documentation when required.

This iterative process can be time-intensive and may involve multiple rounds of review. In addition, every permit application is subject to municipal fees, which vary depending on the scope of work, construction valuation, or square footage affected. The professional's contract with the owner should therefore explicitly define the extent of their responsibility for permitting services and include provisions for compensation covering preparation time, revisions, coordination with municipal personnel, and associated fees.

It is also important to note that multiple permits may be required for a single façade project due to the involvement of specialized subcontractors or consultants. For example, separate permits may be necessary for scaffolding and access equipment, electrical modifications, masonry or stone restoration, structural retrofits, or roofing-related work. Because each permit carries its own cost and administrative burden, responsibilities for obtaining them must be clearly assigned - whether to the owner, the prime professional, or individual subcontractors.

In some cases, certain permits require face-to-face meetings with municipal officials or additional technical reviews prior to approval. When these activities fall under the scope of services expected from the professional, the corresponding efforts and costs must be explicitly stated in the professional services agreement and itemized as part of the project budget.

Liability insurance – Professionals engaged in façade inspection and rehabilitation projects must maintain adequate liability insurance to protect against potential claims or lawsuits arising from their work. The type and extent of coverage depend on the professional's employment arrangement and business structure.

When the professional is employed by an established engineering or architectural firm, liability protection is typically provided through the firm's corporate insurance policy, often structured as an umbrella policy covering all employees engaged in professional services. In such cases, the firm's Human Resources (HR) or Risk Management department is the appropriate point of contact for understanding the firm's liability coverage, determining its limits and exclusions, and obtaining a formal certificate of insurance when required.

For professionals working within a Professional Corporation (PC) structure, responsibility for maintaining individual liability insurance may rest with each licensed professional. The extent of this responsibility is dictated by the firm's bylaws, corporate policies, and applicable state laws (see later discussion on PCs).

Regardless of corporate structure, most clients - particularly governmental agencies at the municipal, state, or federal level - require documented proof of professional liability insurance prior to executing a service contract. The absence of such coverage is often a disqualifying factor in public-sector procurement.

For sole proprietors or small business owners, liability insurance is equally critical. Coverage should extend to all licensed engineers, architects, and technical staff working under the professional's supervision. The scope of coverage must adequately account for risks associated with structural assessments, façade access systems, scaffolding, and any design recommendations or specifications issued as part of the project.

Professional societies and trade associations often provide guidelines on appropriate levels of insurance coverage, tailored to the size of the practice and the scope of services offered. These resources can be especially valuable for sole proprietors and small firms determining minimum coverage thresholds to remain competitive while ensuring financial protection. See the definition of a "small business" discussed later in this chapter.

Contract laws and limits on liability and responsibilities – The extent of responsibilities should be clearly spelled out in the contract. In other words, the contractor (the professional) and the owner (client) know exactly what tasks will be performed and for what total cost. If the owner asks the professional for assistance to provide a bid document, obtain permits, perform supervision of repair works, etc. (in addition to the main task of façade inspection), it is important to make sure these tasks are included in the contract. The services of an expert may be needed when preparing a bid document. Samples of contract forms and templates are available in web sites of several professional societies. The contract must also indicate the limits of the professional's liability.

Management of a business – The daily operation of an engineering or architectural practice requires disciplined management practices that directly influence the reputation and credibility of the professional and the firm within the industry. Adherence to sound management protocols not only ensures compliance with professional standards but also provides a clear audit trail in the event of disputes, claims, or regulatory reviews.

At a minimum, the following areas demand consistent and careful attention:

- Comprehensive documentation and record-keeping: All project-related documents must be accurately prepared, dated, and archived. This includes calculation sheets with clear indication of the preparer, reviewer, and approver, as well as all technical drawings and revisions. Documentation should also cover change orders, addenda, and any modifications to the scope of work.
- Systematic filing procedures: A structured filing system - whether digital or physical - should be maintained for each project. Files must be organized in a manner that allows efficient retrieval and traceability of decisions, calculations, and approvals.
- Communication records: All forms of communication, including emails, letters, meeting minutes, and summaries of important

phone conversations, should be preserved in the project file. These records provide critical evidence of client directives, agreements, and clarifications.
- Schedule compliance: The professional must diligently follow the schedule of activities as defined in the original proposal, contract, and project planning documents. This ensures accountability and helps manage client expectations regarding deliverables and timelines.
- Progress reporting: Regular progress reports should be prepared and delivered in a timely manner. These reports document milestones, identify potential risks or delays, and demonstrate proactive project management.

By consistently applying these practices, professionals strengthen their reputation, mitigate legal and financial risks, and ensure that their work meets both technical and contractual obligations.

Relation with employees and staff members – Whether staff members are directly employed by the professional in a privately owned business entity or report to the professional as part of a larger corporation, it is essential to establish and maintain transparent policies regarding compensation, benefits, working hours, leave entitlements, and company-wide procedures. Clear communication of these policies promotes trust, reduces disputes, and strengthens professional integrity.

Key expectations and obligations toward employees include:

- Professional growth and development: Employees should be provided with opportunities for advancement within the organization. This may include access to mentorship, structured career pathways, and performance-based recognition.
- Training and apprenticeship: Apprenticeship programs and continuous professional development (CPD) are critical for ensuring staff remain current with industry standards, new technologies, and regulatory changes. Field staff, in particular, must receive comprehensive training to carry out inspection and rehabilitation work safely and competently.
- Insurance and benefits: Adequate insurance coverage, including health, liability, and workers' compensation, must be provided in line with statutory requirements and industry best practices.
- Safety provisions: Appropriate personal protective equipment (PPE) and job-specific safety gear must be issued. Employees should also be trained in occupational safety standards, such as those required by OSHA, to reduce risk during site activities.

As the company expands, the administrative complexity of workforce management also increases. At this stage, it is advisable to retain the services of a qualified accountant or human resources professional to manage payroll, employee benefits, tax compliance, workers' compensation, and record-keeping. This ensures that legal obligations are met and employees' rights are safeguarded, while allowing the professional to maintain focus on technical and project-related responsibilities.

Consulting – Whether a professional retains the services of outside experts or provides consulting services themselves, establishing a clear, well-structured working relationship is essential to ensure technical quality, accountability, and cost control. When engaging a consultant, several fundamental steps must be followed:

1. Verification of credentials: Confirm that the consultant holds the necessary professional licenses, certifications, and insurance coverage required by law and by the scope of the project. Request documentation of professional liability (errors and omissions) insurance and general liability insurance to mitigate potential risks.
2. Assessment of expertise: Review the consultant's portfolio of past projects and obtain references, with emphasis on projects of comparable scope, complexity, and technical discipline. Evaluate their familiarity with applicable building codes, safety standards (e.g., OSHA regulations), and municipal permitting requirements relevant to the project.
3. Defining scope of services: Prior to execution of a contract, develop a written scope of work that clearly identifies the services required, technical deliverables, reporting formats, and deadlines. Specify whether the consultant is expected to provide field services, analytical support, design documentation, or expert review.
4. Fee Structures and cost predictability: Negotiate the fee structure upfront. Fixed-fee or lump-sum contracts are often preferable to hourly billing in order to provide cost predictability and minimize disputes. For projects with uncertain scope, a not-to-exceed cap on hourly billing may be used, with clear provisions for scope changes. When providing consulting services, professionals must adhere to the same principles of clarity and accountability:
5. Documentation of scope: Clearly define the tasks, deliverables, and technical outputs in the contract or proposal. Indicate reporting requirements, expected attendance at site visits, and any intermediate submittals.
6. Participation in meetings and coordination: If the client requires attendance at meetings with owners, contractors, or regulatory authorities, the contract should explicitly state the level of participation expected. Fees for meeting attendance, preparation of minutes, and follow-up actions should be itemized separately to avoid later disputes.
7. Professional standards: Both consultants and professionals providing consulting services are expected to uphold the highest standards of technical accuracy, impartiality, and ethical conduct, consistent with professional society codes of ethics (e.g., ASCE, AIA).

By following these structured steps, professionals can safeguard project outcomes, manage risks effectively, and maintain a transparent and mutually respectful relationship with all parties involved.

Relation among all parties involved – Clear, consistent, and timely communication is critical to the success of any engineering or construction project. All parties must have access to accurate information, updated schedules, and clearly defined responsibilities to avoid misunderstandings and delays.

The key stakeholders typically include:

- The Professional of record (engineer or architect), responsible for technical accuracy, code compliance, and overall design integrity.
- The owner, who establishes project objectives, provides funding, and approves major decisions.
- The general contractor, who oversees construction execution, workforce management, and scheduling.
- Subcontractors and specialists, who perform specific tasks and bring specialized expertise to the project.

The professional, often serving as the project manager, plays a central role in coordinating these stakeholders by:

- Facilitating regular meetings; organizing progress meetings at appropriate intervals to review schedules, submittals, change orders, and technical issues; and preparing and distributing meeting agendas and minutes to ensure accountability.
- Establishing communication protocols; and defining formal channels of communication (e.g., Requests for Information, submittals, and transmittals) to ensure traceability of technical and contractual decisions.
- Utilizing project management software or shared digital platforms for document control, version tracking, and distribution of drawings or specifications.
- Ensuring alignment of objectives; and confirming that all stakeholders have a consistent understanding of project scope, performance criteria, milestones, and deliverables.
- Monitoring compliance with contract documents and technical specifications during all phases of the project.

By maintaining structured communication protocols and a disciplined coordination process, the professional strengthens collaboration, minimizes disputes, and supports the timely and cost-effective delivery of the project.

If the professional is also responsible for overseeing the contractor's work, they must request periodic progress reports and ensure regular site visits are conducted. A consistent on-site presence is essential to verify that delivered materials meet specifications, change orders are properly managed, and work quality is upheld.

Record-keeping – All documents associated with a contract or project must be organized, labeled, and stored in a manner that ensures they are readily retrievable. This includes, but is not limited to:

- Contracts, change orders, and addenda.
- Engineering analyses, design calculations, and computer-generated outputs.
- Technical drawings, sketches, and markups.
- Field reports, inspection logs, and progress reports.
- Meeting minutes, memoranda, and correspondence (letters, emails, and text records when applicable).
- Photographic documentation, including dated images of site conditions, construction progress, and any observed deficiencies.

To protect the integrity and accessibility of these records, the following practices should be observed:

- Proper file organization and labeling: All documents should be systematically labeled with project identifiers, dates, and author/reviewer information. A standardized filing system (digital and/or physical) should be established to maintain consistency across projects.
- Redundancy and backup: Critical documents must be duplicated and stored in secure, separate locations, whether through cloud-based systems, external drives, or institutional archiving services. Backup systems should be tested periodically to ensure recoverability.
- Legal and contractual considerations: Records should be retained in accordance with applicable statutes of limitation, municipal regulations, and professional liability requirements. Documentation should be preserved in anticipation of potential disputes, claims, or litigation, whether the professional or their organization is directly or indirectly involved.
- Access control and security: Sensitive documents must be protected against unauthorized access, with appropriate permissions set for digital files and secure storage for physical records. Document revisions should be tracked, with clear identification of authorship and approval at each stage.

Proper document management is not only a best practice in professional engineering and architectural services but also a critical safeguard in defending against legal claims, demonstrating due diligence, and ensuring continuity of project knowledge.

Rules on starting a business entity – For professionals operating as sole proprietors or small business owners, selecting an appropriate business structure is a critical step in managing liability, taxation, and long-term stability. One of the most common and practical forms of business registration is the Limited Liability Company (LLC). The LLC structure provides a relatively simple framework for establishment while offering protection of personal assets against lawsuits and business-related claims. Under this arrangement, the professional is recognized as the owner of a legal entity distinct from their personal affairs, though it is not classified as a "corporation" in the traditional sense.

By contrast, if a company is formally registered as a corporation, the owner becomes legally separated from the company and assumes the role of a shareholder. This structure may provide enhanced liability protections but generally requires greater administrative effort and regulatory compliance.

Given the legal implications of business formation, it is strongly recommended that professionals consult with an attorney experienced in corporate and business law. Legal counsel can guide the selection of the most appropriate entity type, assist in interpreting the statutory obligations, and prepare the necessary filings and documentation for proper registration.

Several business entity types are available to professionals, each with distinct advantages and limitations:

Limited Liability Company (LLC)

- Simplest to establish and manage.
- Provides liability protection by separating personal and business assets.
- Offers flexibility in taxation and ownership arrangements.

Limited Liability Partnership (LLP)

- Designed for businesses with multiple partners.
- Each partner is shielded from liability arising from the misconduct or negligence of other partners.
- Common in professional practices such as law, engineering, or accounting.

S-Corporation (S-Corp)

- Provides limited liability protections similar to a corporation.
- Treated as a "pass-through" entity for tax purposes, avoiding corporate-level taxation.
- Well-suited for small to medium-sized professional practices seeking both liability protection and favorable tax treatment.

Limited Partnership (LP)

- Comprises at least one "general partner" who manages operations and assumes liability, and one or more "limited partners" whose liability is restricted to their investment.
- Less common in professional engineering or architectural practices but may be applicable in certain investment-driven projects.

Professional Corporation (PC)

- Typically formed by licensed professionals such as engineers, architects, lawyers, doctors, or accountants.
- Provides a corporate framework, but each professional remains independently liable for their own professional actions.
- Requires individual liability insurance coverage for each shareholder-professional.

In summary, for an individual professional or sole proprietor, establishing an LLC is generally the most straightforward and advantageous option, offering both liability protection and operational flexibility. For small practices involving multiple professionals, an LLP or an S-Corporation may provide better alignment with shared ownership and liability management.

The ultimate choice of business entity should be made with careful consideration of the firm's size, ownership structure, growth objectives, and regulatory requirements. Legal and financial advisors should be consulted early in the process to ensure compliance and optimal structuring of the practice.

Small business concern – A Small Business Concern (SBC) is defined as an enterprise with a limited workforce and operational

scale, formally recognized under the U.S. Small Business Act. Whether a company qualifies as an SBC is determined by several regulatory criteria established by the Small Business Administration (SBA, 2016). These criteria are particularly important when pursuing government procurement opportunities, applying for federal assistance programs, or competing for contracts set aside for small businesses.

According to the SBA, a business must meet the following requirements to be classified as an SBC:

Ownership and control: The enterprise must be independently owned and operated. It cannot be a subsidiary or a controlled entity of a larger corporation.

Market position: The business must not be dominant within its field of operation, ensuring competitive balance within the industry sector.

Size standard compliance: The company must not exceed the SBA's small business size standards, which vary depending on the type of industry and the specific procurement action. These standards are typically based on the number of employees or average annual receipts.

Location and operations: The business must be located in the United States and conduct its primary operations domestically.

Economic contribution: The business must contribute to the U.S. economy, either by paying taxes, utilizing American products and materials, or employing U.S. labor.

General workforce threshold: In most industries, the maximum number of employees permitted under the SBC designation is 500 employees, though the threshold can vary depending on the pertinent North American Industry Classification Code (NAICS).

Eligible business forms: An SBC can be structured as any for-profit entity, including but not limited to:

- Corporation
- Partnership
- Limited Liability Company (LLC)
- Sole proprietorship
- Joint venture (with less than 50% foreign participation)
- Association
- Trust
- Cooperative business venture

In summary, the SBC designation plays a critical role in enabling smaller enterprises to compete on an equitable basis for government and private-sector contracts. For professionals in engineering, architecture, or construction, ensuring compliance with SBC criteria not only affects eligibility for certain projects but also provides competitive advantages in markets where size standards are strictly enforced.

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5.8: Exercises

E5.1 Which of the following statements best describes the definition for “ongoing inspection” as used by some municipalities?

a) An inspection that is called for after a reported damage.

b) An inspection conducted halfway into the interval for a critical examination.

c) An inspection that had to be terminated because of weather condition.

d) An inspection that was delayed because of legal issues.

E5.2 Which of the following is least likely to be considered a critical façade inspection?

a) An inspection conducted thoroughly aboard a moveable scaffold on all façades.

b) An inspection conducted for a 90-foot-high (27.4-meter-high) building over at least a 4-year interval in Chicago.

c) An inspection conducted in New York City for an 8-story building over a 5-year interval.

d) An inspection done annually, as a follow-up, to make sure the condition of the façade is safe.

E5.3 Who is considered a qualified exterior wall inspector in New York City?

a) A registered professional engineer or architect with additional training and certification required for façade inspection in New York.

b) Any registered professional engineer or architect licensed to practice in the State of New York.

c) Any professional with engineering or architecture education and at least a BS degree.

d) Any individual with several years of experience in façade inspection.

E5.4 In compiling data as part of planning for façade inspection of a building, which of the following sources, in your opinion, would be most useful in finding out about the façade inspection ordinance in a major urban city?

a) The specification page (the lead drawing) in the set of architectural drawing of the building.

b) An old inspection report for the façade inspection of a similar building.

c) A copy of the permit issued by the city for a prior façade inspection.

d) The building code used by the city.

E5.5 Which of the following statements is least accurate?

a) Most major cities in US, with façade inspection ordinances, have their own code for buildings.

b) Not all major cities in US, with façade inspection ordinances, have their own code for building.

c) The three major cities in Ohio have their own ordinances for buildings.

d) None of the major cities in US, with façade inspection ordinances, adopt any of the national building codes.

E5.6 What are major differences when a professional registers their company as an LLC or as a corporation?

a) In both options, the professional, as the owner, works for a group of shareholders.

b) In an LLC setting the professional is the company; while in a corporation, the professional is separated from the company and is a shareholder.

c) In a corporation, the professional is the sole owner; while in an LLC, they are separated from the company holding shares.

d) In both options, the professional is among several shareholders, who each work independently.

E5.7 In a company a group of engineers and architects work independently and each carries their own liability insurance. Which of the following designation best describes this company?

a) It is a professional corporation.

b) It is a limited liability company.

c) It is a limited liability partnership.

d) It is a limited partnership.

E5.8 Which statement is least accurate?

a) A liability insurance is often required before you engage in a contract especially with most government agencies.

b) Any sole proprietor needs to have insurance coverage for protection against potential lawsuits.

c) Engineers who work as employees of a corporation doing façade inspection work are generally covered under an umbrella liability insurance, where the company is the policy holder.

d) Only engineers who work for themselves need to have liability insurance coverage.

E5.9 Why is it important to know about the structural condition of the roof of the building for which façades are scheduled for a critical inspection?

a) To make sure there are no large air conditioning units hindering inspection activities.

b) To make sure structural members are strong enough for workers walking on roof.

c) To make sure structural members are adequate for any roof-installed anchors.

d) To prepare new structural drawings for the façade inspection.

E5.10 What is the purpose of inspectors going through scaffold safety training?

a) To get familiar with OSHA rules on safety when working on scaffolds.

b) To have proper credentials that are needed by municipalities when an inspector is at the site working on a scaffold.

c) To know the rules regarding the safety lifeline and requirements for proper installation of scaffolds.

d) All of the above.

E5.11 Which of the following statements is least accurate?

a) Before inspection, it is important to identify the types and location of ancillary façade elements such as balconies, fire escapes, flagpoles, canopies, etc., to ascertain the accessibility of areas on the exterior walls.

b) Scaffolds are often needed in critical inspection of façades because most municipalities mandate close examination of façade elements.

c) All municipalities in North America that have mandates for façade inspection accept a previous inspection report in lieu of a new one, if a general contractor indicates there has not been any changes since the last critical inspection.

d) Before doing the inspection, it is important that all personnel involved in the inspection process understand the required safety rules and have proper training and certifications as needed.

E5.12 Which of the following statements is least accurate?

a) When you hire employees to work for you in your company, it is important that they are briefed and well familiar with the policies of the company including benefits, insurance coverage, etc.

b) To be qualified as a small business concern, generally a company cannot have more than 500 employees.

c) The knowledge of contract laws, which concern the extent of responsibilities and liabilities when entering into a contract, is only important to large projects.

d) The services of an accountant are often needed to process employees' payroll, taxes, etc.

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6: PARAPET WALLS, ROOFS AND ROOF INSTALLATIONS FOR SCAFFOLDS, GREEN ROOFS AND WALLS

Chapter Synopsis

In this chapter, condition assessment and damage issues of parapet walls and roofs are covered. Roof structural members often serve as support for anchors that are needed for safety lines, moveable scaffolds and platforms, used in façade inspections. The safety issues and design requirements for these anchors are covered in this chapter including those recommended by the Occupational Safety and Health Administration (OSHA). Green roofs and green walls are also covered with information on their composition, function and relation to building envelope inspection.

6.1: Introductory Remarks

6.2: Parapet Walls

6.2.1: Structural engineering considerations

6.2.2: Parapet wall inspection process and distress conditions

6.3: Roofs

6.3.1: Roof Terminologies

6.3.2: Roof distress conditions

6.3.3: Structural considerations

6.4: Green Roofs

6.4.1: Green roof classification

6.4.2: Green roof layers

6.4.3: Green roof loads

6.4.4: Green roofs within the context of envelope inspection

6.5: Green Walls

6.5.1: Kinds of green walls

6.5.2: Benefits, environmental impacts, distress conditions, and mitigations

6.5.3: Structural considerations

6.5.4: Economic considerations

6.6: Chapter References

6.7: Exercises

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6.1: Introductory Remarks

This chapter addresses the following key elements of building envelope inspection:

- Parapet walls – their functions, structural behavior, and common distress conditions.
- Roofs – their role as load-bearing systems and as platforms for supporting safety lines, suspended scaffolds, and inspection equipment.
- Green roofs and green walls – their composition, functions, and specific considerations in façade inspection.

Parapet walls: Parapet walls are the upward extensions of a building façade beyond the roofline and act as barriers at roof edges. They are critical to both safety and building performance, and therefore form an integral part of façade inspections. When evaluating parapet walls, the primary areas of concern include:

- Structural stability and integration – ensuring proper connection with the façade wall and roof structure to prevent separation or collapse.
- Load resistance – verifying adequate strength against wind loads, seismic forces, and other lateral or vertical loads as prescribed by building codes.
- Architectural detailing and waterproofing – reviewing design elements such as flashing, coping, and roof junctions to ensure continuity of waterproofing and protection against infiltration.
- Fire protection – confirming compliance with fire-rating requirements where parapet walls serve as fire barriers.
- Aesthetics and durability – assessing weathering, material degradation, and overall contribution to façade appearance.

Roofs: Roofs are a fundamental part of the building envelope and must be considered in conjunction with façade inspections. Problems originating at the roof - such as membrane deterioration, flashing failures, or ponding effects of water - often migrate to the parapet walls and façades. Therefore, roof inspections should be explicitly included in the project scope and contract documents submitted to the building owner.

From a structural perspective, roof systems may also serve as anchor points for suspended scaffolds, rope descent systems, and fall-protection equipment. In such cases, evaluation by a licensed structural engineer may be required to ensure that:

- Parapet walls can resist the code-prescribed lateral loads without risk of failure.
- Roof beams, trusses, and decking are capable of supporting OSHA-mandated loads associated with anchors, lifelines, and suspended work platforms.

The chapter also reviews the architectural and structural details of parapet walls and roofs, focusing on loading conditions that may influence their long-term integrity. These include:

- Mechanical loads from safety anchors, scaffolding, and suspended platforms.
- Environmental loads such as snow, wind uplift, and temperature-induced expansion/contraction.
- Green roof loads, including soil, vegetation, irrigation systems, and retained water.

Special emphasis is placed on the analysis of anchorage installations for safety systems to ensure compliance with OSHA requirements and structural adequacy under multi-directional load applications.

Common distress conditions are also covered in this chapter. These include:

- Cracking, displacement, and loss of masonry units in parapet walls.
- Deterioration of coping stones and flashing at roof–parapet junctions.
- Membrane punctures, blisters, and water intrusion in roof systems.
- Structural deterioration of roof framing members due to overloading, corrosion, or long-term deflection.
- Vegetation- and moisture-related issues specific to green roofs, such as root penetration or drainage failure.

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6.2: PARAPET WALLS

Parapet walls are barrier walls at the edge of a roof. Often, they are the extension of exterior walls. Different kinds of parapet walls can be recognized by their construction types or materials used. In terms of construction type, parapet walls can be categorized as follows

- Continuous solid.
- With openings.
- With a stepped appearance.
- Embattled (which are also called castle types).

Solid parapet walls are among the most popular ones used in variety of applications. In terms of material types, parapet walls mainly follow the same materials used in their façade walls. They can be made up of all concrete, brick, CMU, other masonry units, or metal.

In terms of appearance and architecture, most solid parapet walls resemble the rest of exterior walls. However, they may also possess their own architectural detailing that includes proper flashing and a coping stone or cap at the top. Figure 6.1 depicts the details of a typical parapet wall. It is, however, noted that there are many variations in architectural detailing and the form and type of flashing used.

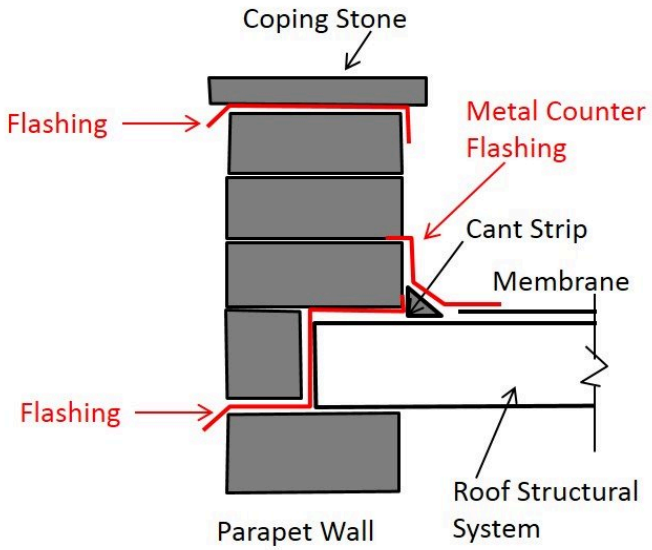


Figure 6.1 The schematics of a solid parapet wall and its connection to the roof

(This figure shows the cross-section of a parapet wall with a coping stone at the top, and three kinds of flashing. One is shown just below the coping stone. A second one is labeled as metal counter flashing, and is situated at the junction of the parapet wall and the roof. A third one is shown between the roofing materials and the wall, where roof extends into the wall.)

The coping stone may have three different shapes depending on whether it is slopped or not. Figure 1.9 (in Chapter 1) shows three different kinds of coping stones. A distinctive feature of a coping stone is having a “drip groove.” The lack of this feature is considered as a deficiency, which needs to be addressed in the inspection report with recommendation to make the necessary corrections.

Figure 6.2 provides several other alternative parapet wall architectures.

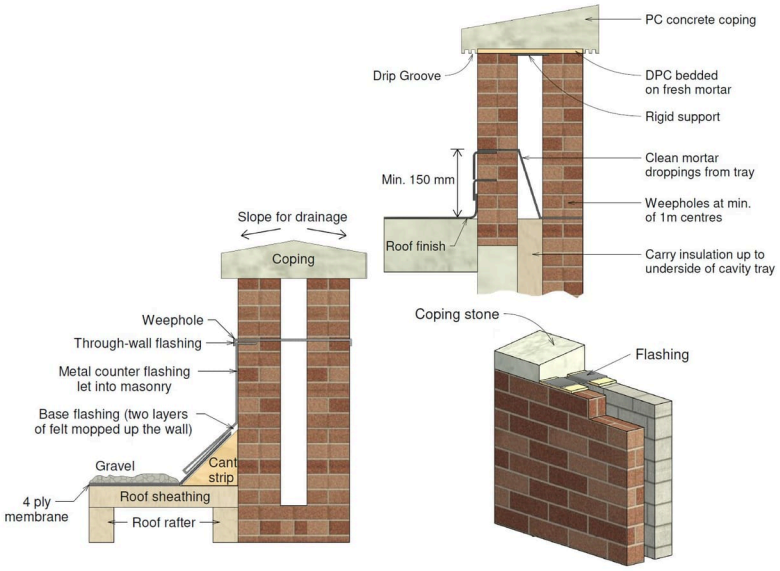


Figure 6.2 Different solid parapet wall architectures

(This figure shows three different details for parapet walls and different coping stones and flashing configurations.)

Figure 6.3 shows a photo of a parapet wall as seen from the roof. In this case, the coping stone in this parapet wall is made up of clay tiles overlapped and sealed properly to divert water migrating into the wall.



Figure 6.3 Photograph of a parapet wall with clay tiles used as coping stones

(This figure shows a photograph of a brick parapet wall from the roof. The metal flashing installed on the wall is visible in the figure. The coping stone is made up of clay tiles.)

Section 705.11.1 of the International Building Code, see IBC (2011), indicates the following requirements for parapet walls.

Parapet walls will require to have the same fire-resistance rating as that required for the supporting wall. On any side adjacent to a roof surface, they are required to have noncombustible faces for the uppermost 18 inches (457 millimeters), including counterflashing and coping materials. The height of the parapet is recommended to be at least 30 inches (762 millimeters) above the point where the roof surface and the wall intersect. Where the roof slopes toward a parapet at a slope greater than two units vertical in 12 units horizontal (16.7-percent slope), the parapet wall needs to extend to the same height as any portion of the roof

within a fire separation distance where protection of wall openings is required, but the height cannot be shorter than 30 inches (762 millimeters).

The requirement for the parapet wall is generally more severe when the roof is used as a deck with people occupancy. Installation of screens on the top of parapet walls may be required by the municipal codes for occupant and pedestrian protection (see Figure 6.4).

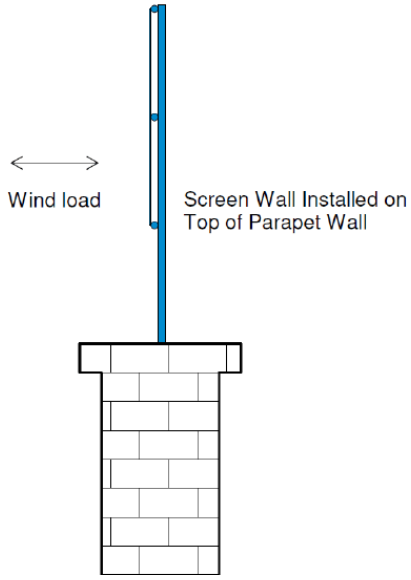


Figure 6.4 A screen installed atop a parapet wall for occupant and pedestrian protection

(This figure shows a cross-section detail of a parapet wall with a screen wall installed on the top. The direction of the wind load is also shown.)

6.2.1: Structural engineering considerations

6.2.2: Parapet wall inspection process and distress conditions

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6.2.1: Structural engineering considerations

There is no specific recommendation for lateral loads applied on parapet walls. However, as any other structural members, they will need to be designed for stability and strength. A major load component is the wind load, which may result in overstresses in, and instability of, the parapet wall. In situations when screens are installed atop a parapet wall, as shown in Figure 6.4, the wind load effect on the wall will be more pronounced.

In addition to the wind load, parapet walls are also vulnerable to rain, snow, ice build-up, and thermal forces, see Kuriyama and Au (2020). Parapet walls will affect the snow pile-up on the roof – a phenomenon called "snow drift." In addition to the increased snow load, the drift contributes to ice and snow build-up, which may cause leak.

If necessary, parapet walls will need to be reinforced for proper strength and stability. Often, parapets require incorporation of steel as reinforcement. Occasionally, parapet walls also support roof vegetations, decks, mechanical equipment, flag poles, ornamental pieces, and screens (as mentioned earlier); and as such, they must be designed and detailed appropriately for the effects from the gravity and wind loads. In these situations, reconstruction may then become necessary with installation of a reinforced concrete bond beam to properly and securely transfer the applied loads to the wall, see Figure 3.4.

Generally, the local code requirements outline the kind of loads that need to be considered for design of parapet walls and appropriate structural detailing acceptable for use. For example, according to the Chicago Building Code (CBC), masonry and reinforced parapets must have at least the same thickness as the required thickness of the wall on which it is provided, but need not be more than 12 inches (305 millimeters) thick on exterior walls, see Section 15-8-100 of CBC (2019). It is therefore advised to hire a structural engineer to properly address the structural design and stability of parapet walls, when they are subject to gravity and wind loads, and to adhere to the local building code requirements in situations when wall repairs and reconstruction or retrofit are required.

Illustrative example – Example 6.1 in Appendix A demonstrates a simple computation for stresses on the mortar joints developed because of wind load on a parapet wall.

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6.2.2: Parapet wall inspection process and distress conditions

The inspection process for parapet walls follows the same requirements as that for façades. In most cases, the inspection is conducted aboard a scaffold and also from the roof. In addition to visual inspections, removal of coping stone may be necessary to investigate the proper installation of flashing. From the roof side, the examination of the detailing and flashing at the junction with the roof also needs attention. Follow-up analyses may need to be conducted to address the effect of wind and other loads in maintaining the strength and stability of the parapet wall.

The following is a list of the kinds of distress conditions to look for during an inspection of a parapet wall.

- Coping stone is damaged; the joint mortar is missing; the drip groove is not present.
- Flashing is missing; on the roof side, flashing detail is not correct (flashing on the wall should properly overlap the roof flashing, see Figure 6.1).
- The wall is bulged in or out (see Figure 2.13 of Chapter 2). By visual observation, staying at one corner on the roof and watching the alignment of the wall is one way to find out whether the wall is bulging in or out.
- Bricks or other masonry units are damaged or missing.
- Mortar joints are damaged and need tuckpointing.
- Efflorescence is present.
- Cracks are present indicating either lack of expansion joints or damaged lintels supporting the wall over openings.
- In long walls, there are no expansion joints.
- The height does not confirm to the code requirement.

Coping stone distress issues – Whatever material used, the coping is a cover for the top of the masonry wall and guards against rain and snow entering the top of the parapet wall. With clay coping tiles, the joints connecting the tile pieces will deteriorate over time, permitting the migration of water into the wall. Non-professional repairs only aggravate the problem. These joints should be repaired with quality mortar and sealant.

The proper installation calls for the stone to have a through-wall flashing underneath, be beveled and sloped (so as to shed water), and to have a drip grove at the edges. As shown in Figure 1.9 (of Chapter 1), this is a small groove, cut under the edges of the stone, which will cause water to drip off and not be absorbed under the stone. If the copings are not properly installed and flashed, water will enter the top of the wall and infiltrate into the interior of the building.

Figure 6.5 shows two photographs of coping stone damage. The photo on the left shows a coping stone that appears to be in good repair. However, upon further investigation, the lack of drip grooves was noted and there was no flashing under the coping stone. The image on the right in the figure shows an application of hard mortar between joists, which has shrunk and thus becoming ineffective.





Figure 6.5 Coping stone damage cases (from personal collection of the lead author)

(This figure shows two photographs. The one on the left shows a brick parapet wall that appears to have been tuckpointed recently with coping stones without drip grooves. The one on the right shows coping stones with shrunk mortar at the joints.)

Cracks – Another common distress condition in parapet walls is the formation of cracks. Crack formation can be attributed to thermal expansion, especially in situations where expansion joints are not present, freeze/thaw cycles, periodic wet/dry cycles, inadequate design, and external loads. Inadequate structural design and detailing for lintels that support parapet walls may also contribute to crack formation. This can be the result of excessive deflections or steady vibrations experienced by a lintel because of air-conditioning units and other mechanical devices running in the proximity of the affected lintel.

Other kinds of distress condition – Moisture damage resulting from inadequate design, lack of, or damaged in, flashings, tear and wear, dislocation of masonry units, and wall deformation are also among other kinds of distress conditions in parapet walls.

Table 6.1 provides a summary of various distress conditions, their probable causes and lines of actions that can be implemented to address them.

Table 6.1 Distress conditions in parapet walls, probable causes, and suggested lines of actions

Distress Condition	Probable cause	
Coping stone is damaged; the joint mortar is missing; the drip groove is not present	Weather; water washout; freeze/thaw; poor workmanship; age	Weather and water c coping tone Poor workmanship a sealant
Flashing is missing; on the roof side; flashing detail is not correct	Weather; poor design; age	Remove coping stor proper detail
Wall is bulged in or out	Strong wind load; poor workmanship	Rebuild wall
Brick is damaged or missing; mortar joints are damaged	Weather; age; water washout; freeze/thaw; age	Replace damaged b
Efflorescence deposit	Low quality brick; persistent moisture present	Replace bricks (sinc quality bricks are ne
Cracks	Lack of expansion joint; inadequate design and detailing of lintels that support parapet walls; thermal expansion and contraction; periodic wet/dry cycles	Reconstruct wall aft installing lintel)
Inadequate wall height	Poor design	Repair and/or rebuil
Fire protection and safety violation	Poor design	Repair and/or rebuil

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6.3: ROOFS

Roofs are also considered as a component of a building envelope system. The significance of the roof during performing an envelope inspection is that it must

- Have structural members that are strong enough to support anchors installed for fall protection and the weight of the scaffold and the scaffold frame system.
- Have proper architectural details at their intersection with the parapet wall.
- Be evaluated when the façade and the parapet wall are found to be damaged because of water leaks; since the roof details and damage (e.g., membrane damage) may be the cause.
- Be inspected to identify and note the existence of equipment that may be obstacles to the façade inspection and installation of scaffold frames.

6.3.1: Roof Terminologies

6.3.2: Roof distress conditions

6.3.3: Structural considerations

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6.3.1: Roof Terminologies

Figure 6.6 presents terminologies used in roofs. Knowing these terms is important when communicating with roof specialists discussing kinds of distress conditions that are prevalent in the roof and the parapet walls.

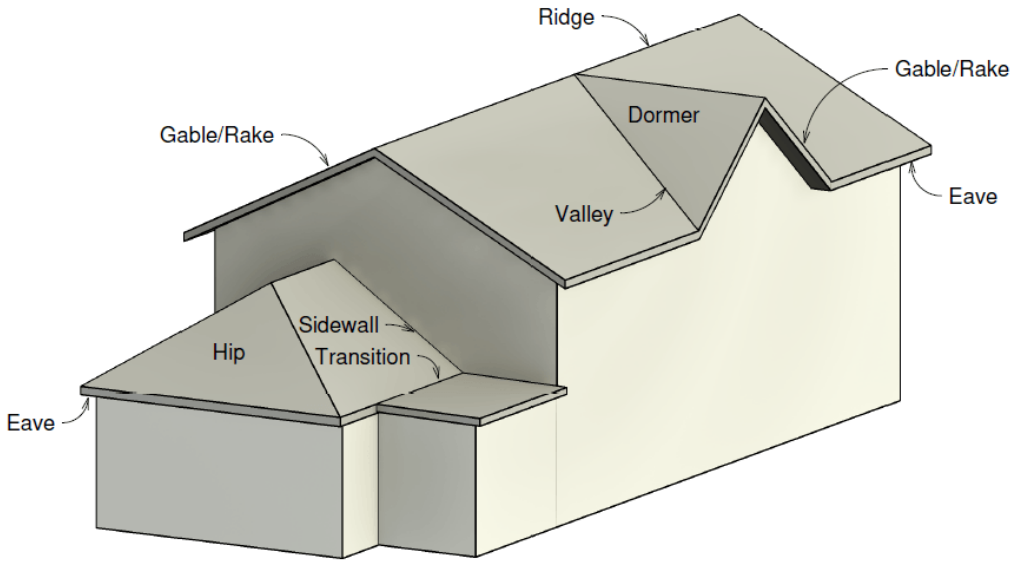


Figure 6.6 Roof terminologies

(An illustration of a sloped roof with common terminologies is shown. The edge is an eave, the top is a ridge, the sloping part is a gable/rake, and the intersection between a sloped and a flat roof is called transition. A smaller raised secondary roof is called a dormer. The intersection of the dormer to the sloping roof is a valley. The vertical surfaces at the ends of the sloping roof are sidewall and hip.)

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6.3.2: Roof distress conditions

Roof distress conditions can be in one or more of the following situations.

Tear in rubber membrane – Ethylene propylene diene monomer (EPDM) rubber is often used in roofs as the membrane. Damage to this membrane is in the form of a tear that will cause leak.

TPO roof coating cracks – TPO stands for "thermoplastic polyolefin," a single-ply membrane made from a polymerized combination of polypropylene and ethylene-propylene rubber. It can be attached to a roof with adhesive or with mechanical fasteners. White TPO roofs offer energy efficiency as well as lightweight roof protection. With age, the TPO coating may crack which may promote further damage. The TPO coating is especially vulnerable to ultraviolet (UV) light. More accelerated TPO damage in areas exposed to more intense UV light because of higher altitudes (such as in Denver, Colorado) has been observed, see also McGroerty and Taylor (2014).

The subject of UV lights in accelerated aging of TPO coating has been the subject of several studies such as that in Taylor and Xing (2015).

Flashing damage – As discussed in the case of parapet wall attachments to the roof, flashing helps water drain properly. The flashing runs over the entire perimeter of the wall, thus, if not done carefully, it will cause water damage to the roof, parapet wall, and façades. Flashing damage is often visible as separated, damaged, or being loose in some areas.

Soft membrane in some spots or appearance of bumps – This type of damage may result from poor installation practices or damage to the roofing membrane itself. One common issue involves the entrapment of water or air beneath the membrane, which can lead to the formation of soft spots on the roof surface. Another possible cause is the presence of loose fasteners – components used to secure the membrane in place – which can compromise the integrity of the roofing system.

In addition to installation-related problems, various weather-related factors can significantly contribute to the deterioration of roofing materials. Berdahl et al. (2008) provide a comprehensive overview of these environmental influences. Among them, extreme weather events such as high winds, flying debris, heavy rainfall, hailstorms, and snow accumulation are particularly damaging. These conditions can accelerate wear and tear, leading to material degradation and potential system failure.

Given these risks, it is essential to perform regular roof inspections. Special attention should be paid following severe weather events to identify any signs of damage early. Timely detection and repair are crucial to maintaining the longevity and performance of the roofing system.

Table 6.2 summarizes distress conditions experienced while inspecting roof, causes, and lines of actions that are recommended for implementation.

Table 6.2 Roof distress conditions

Distress type	Probable cause	Lir
Tear in rubber EPDM Membrane	Wear and tear; weather; age; accidental damage while working on roof; severe storms causing damage from hail and flying debris	Minor tears: repair by using pat Extensive tear and damage on c
TPO Roof Coating Cracks	Age, wear and tear; sunlight (UV light); extreme weather conditions	Replace coating
Flashing damaged or missing	Extreme weather conditions; age; poor workmanship; inadequate detail	Replace old deteriorated flashir proper detail

Membrane appears soft in some spots or has bumps	Poor installations, membrane damage causing water or air bubbles trapped underneath membrane; loose fasteners (that are used to secure the membrane)	Minor tears: repair by using pat Extensive tear and damage on c
Roof deformation, partial failure	Excessive deformation and damage in roof structural members because of heavy snow load or mechanical loads	Strengthen structural members

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6.3.3: Structural considerations

In certain cases, additional structural analysis of the roof may be warranted—particularly when damage to parapet walls or active leaks is observed. These issues often become apparent during late winter or early spring, when the accumulation of snow and ice can place significant stress on the roof system. During inspections, visible structural damage may indicate the presence of excessive external loads. These loads can result from a variety of sources, including snow accumulation and drifting, the weight of vegetation and soil in green roof systems, or temporary installations such as scaffolding and platforms used for façade inspections, repair work, or window cleaning. While damage resulting from such heavy loading conditions may fall outside the specific scope of a façade inspection contract, it is essential for the architect or engineer of record to document any concerning observations. They should also advise the building owner on whether further investigation or structural analysis is necessary to evaluate the roof's integrity and ensure continued safety and performance.

Mechanical loads from installation of suspended platforms – Roof structural members are commonly used as anchorage points for suspended platforms, scaffolding, and fall protection systems during maintenance or construction activities around a building. In such cases, it is essential to analyze and evaluate the strength and stability of these structural members to ensure they can safely support the applied loads. When anchors for fall protection or suspended platforms are installed, the roof structure must be capable of withstanding an additional concentrated load of 5,000 pounds (22.2 kilonewtons) applied in any direction, as specified by OSHA (2016).

When a 5,000-pound (22.2-kilonewton) load is applied to a structural roof beam – as is typical for anchor points used in fall protection systems – the primary internal forces generated include a combination of biaxial bending moments and torsion. These complex loading conditions can be particularly challenging for individual beams, especially if they possess low lateral bending or torsional stiffness. In such situations, a single beam may not be sufficient to resist the applied forces effectively. To enhance structural performance and ensure safe load transfer, the installation of additional lateral or torsional bracing between adjacent beams should be considered. This approach promotes better load distribution and increases the overall stability of the system. To ensure compliance with safety regulations and structural integrity requirements, it is strongly recommended that a licensed structural engineer be consulted to assess the suitability of the roof framing and design appropriate reinforcement measures.

Snow load – Knowing the snow load and its profile on the roof is important when investigations into the structural resistance of roof beams are conducted. This will be the case when excessive deformation and damage in roof structural members are suspected as the cause for roof distress conditions. Because of the existence of the parapet walls and the significance of neighboring buildings, there is a possibility for snow drift, which will cause a change in the snow load profile on the roof and also may cause ice and snow build-ups. In this section, an overview of snow load computation on roofs is presented. These materials are taken from ASCE-7 (ASCE, 2022).

The basis of snow load intensity at the roof is the "ground snow load." Depending on various geometrical parameters, this value is adjusted to represent the roof snow load, and the potential for snow drift because of the parapet wall or upper and lower roof configurations. Many factors are involved in snow load computations. Table 6.3 presents a summary of these factors and their explanations.

Table 6.3 Parameters involved in snow load computation

	Parameters

C_e = exposure factor as determined from Table 7.3-1 of ASCE-7

C_s = slope factor as determined from Figure 7.4-1 of ASCE-7

C_t= thermal factor as determined from Table 7.3-2 of ASCE-7

h = vertical separation distance in feet (meters) between the edge of a higher roof including any parapet and the edge of a lowe

h_b = height of balanced snow load determined by dividing p_s by g, in feet (meters)

h_b = clear height from top of balanced snow load to (1) closest point on adjacent upper roof, (2) top of parapet, or (3) top of a pr

h_c = height of upper roof in feet (meters) measured from the level of balanced snow load in lower roof, as shown in Figure 6.7

h_d = height of snow drift, in feet (meters)

h_{d1} or h_{d2}= height of snow drift where two intersecting snow drifts can form, in feet (meters)

h_0 = height of obstruction above the surface of the roof, in feet (meters) – if parapet wall height is h_r, then $h_0 = h_r$

l_u = length of the roof upwind of the drift, in feet (meters)

p_d = maximum intensity of drift surcharge load, in pounds per square foot (kilonewton per square meter)

p_f = snow load on flat roofs (“flat” means a roof slope less than or equal to 5 degrees), in pounds per square foot (kilonewton pe

p_g = ground snow load as provided in pertinent graphs and tables in ASCE-7; or from a site-specific analysis, in pounds per squa

p_m = minimum snow load for low-slope roofs, in pounds per square foot (kilonewton per square meter)

p_s = sloped roof (balanced) snow load, in pounds per square foot (kilonewton per square meter)

s = horizontal separation distance between the edges of two adjacent buildings, in feet (meters)

S = roof slope run for a rise of one

q = roof slope on the leeward side, in degrees

w = width of snow drift, in feet (meters)

w_1 or w_2 = width of snow drift where two intersecting snow drifts can form, in feet (meters)

W = horizontal distance from eave to ridge, in feet (meters)

W_2 = winter wind parameter, which is the percent time wind speed is above 10 mile per hour (4.6 meters per second) during wir
pertinent graphs and tables in ASCE-7

g = snow density, in pounds per cubic foot, per ASCE-7 Equation 7.7-1. The SI unit will be kilograms per cubic meter.

ASCE-7 provides the ASCE Design Ground Snow Load Geo-database or a graphical representation of the data shown in Figure 7.2-1A through 7.2-1D of ASCE-7 for the ground snow loads, p_g, to be used in the determination of design snow loads.

For example, for Northern Illinois (Chicago), the value is 36 pounds per square foot (1.7 kilonewtons per square meter) for Risk Category I and 76 pounds per square foot (3.6 kilonewtons per square meter) for Risk Category IV, respectively.

Furthermore, as indicated in Table 6.2, Parameter W_2 is obtained from graphs and tables in the ASCE document. Generally for regions west of the Rockies and in Southwest, this parameter is around 0.25 to 0.45; while in the Midwestern and Northwest regions, the parameter is around 0.45 to 0.65.

Flat roof snow load – The flat roof snow load, p_f, is calculated from the following equation.

$$p_f = 0.7 C_e C_t P_g \tag{6.3.3.1}$$

Minimum snow load for low-slope roofs – A minimum roof snow load, p_m, is only used for mono-slope, hip and gable roofs with slopes less than 15 degrees, and to curved roofs where the vertical angle from the eaves to the crown is less than 10 degrees. The minimum roof snow load for low-slope roofs is as follows:

Where p_g is equal to or less than the value of the minimum snow load upper limit, $p_{m,max}$, shown in Table 7.3-4 of ASCE-7, then $p_m = p_g$

Where p_g exceeds the value of the minimum snow load upper limit, $p_{m,max}$, shown in Table 7.3-4 of ASCE-7, then $p_m = p_{m,max}$

Sloped snow load – ASCE-7 recommends that snow loads acting on a sloping surface will need to be assumed to act on the horizontal projection of that surface. The sloped roof (balanced) snow load, p_s, is obtained by multiplying the flat roof snow load, p_f, by the roof slope factor, C_s, i.e.,

$$p_s = C_s p_f \tag{6.3.3.2}$$

For leeward drifts, the drift height, h_d, is determined directly from the equation given in ASCE-7, and provided below, using the length of the upper roof.

For windward drifts, the drift height is determined by substituting the length of the lower roof for l_u in the same equation, however, using three-quarters of h_d as the drift height. Following these, therefore, for the leeward drift, using feet as the unit, the drift height h_{dL} is

$$h_{dL} = 1.5\sqrt{\frac{p_g^{0.74} l_u^{0.7} W_2^{1.7}}{\gamma}} \tag{6.3.3.3}$$

And, for the windward drift, using feet as the unit, the drift height h_{dW} is

$$h_{dW} = 0.75[1.5\sqrt{\frac{p_g^{0.74} l_u^{0.7} W_2^{1.7}}{\gamma}}] \tag{6.3.3.4}$$

It is also noted that that for the leeward drift, l_u is the length of the upper roof and h_{dL} need not be taken larger than 60% of the length of the lower-level roof; whereas for the windward drift, the value for l_u is the length of lower roof. The larger value obtained for these two heights is recommended to be used in design as the final value for h_d, i.e, h_d = larger of h_{dL} and h_{dW}, see Figure 6.7.

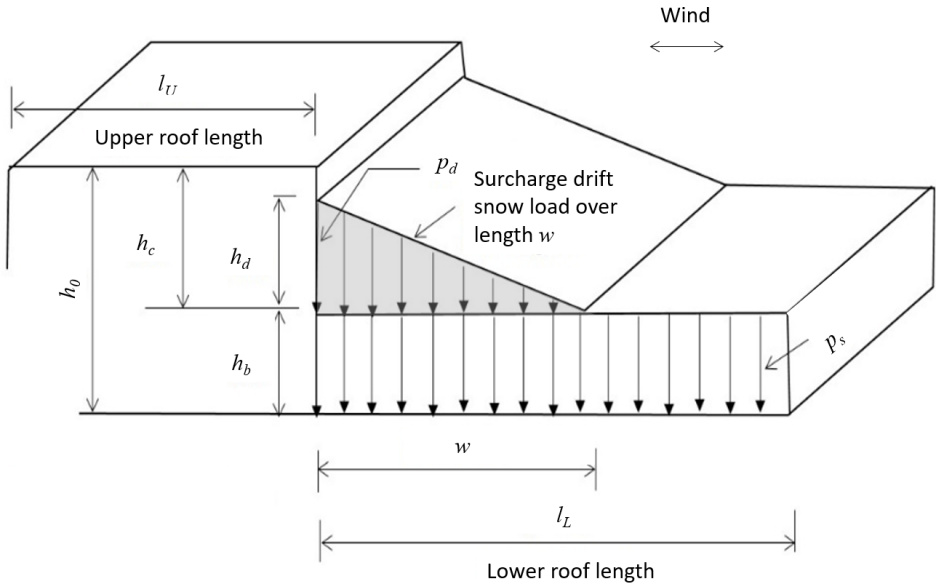


Figure 6.7 Snow load profile, ASCE (2022)

(This figure shows the details of snow drift profile on a roof. It shows a gradual increase of snow load from the balanced snow load to a maximum drift load over a distance labeled as w. Parameters that determine the profile are also labeled on the detail.)

Drift length – For the drift width, w, the following conditions are considered:

For the leeward drift, if $h_{dL} \leq h_c$, then $w = 4h_{dL}$, where the drift height is equal to h_{dL}. However, if $h_{dL} > h_c$, then $w = 4\frac{{h_{dL}}^2}{h_c}$, where the drift height is equal to h_c. In this case, the drift width shall not be greater than $8h_c$.

For the windward drift, the drift width is equal to $8h_{dW}$.

Exceptions:

- If the height of the parapet wall is taken as h_r, then $h_c = h_r - h_b$, per description given for h_c and as shown in Figure 6.7.
- If the drift length, w, for either windward or leeward drifts exceeds the length of the lower roof, the drift is truncated at the far edge of the roof, not reduced to zero there.

It is also noted that the windward and leeward drifts must be checked independently to determine the one that controls the design of affected roof structural members.

After computing h_d, the maximum intensity of the drift surcharge load (p_d) is computed from the following equation:

$$p_d = h_d\gamma \tag{6.3.3.5}$$

In which γ is snow density which is computed as follows:

$\gamma = 0.13p_g + 14$ but not more than 30 pounds per cubic foot ($\gamma = 0.426p_g + 2.2$ but not more than 4.7 kilonewtons per cubic meter).

Illustrative examples – Examples 6.2 and 6.3 in Appendix A illustrate the procedure for computing snow load and drift intensity and profiles.

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6.4: GREEN ROOFS

The incorporation of gardens and vegetation on roofs, commonly referred to as “green roofs,” imposes significant additional demands on the structural members of the roof. Moreover, specialized architectural detailing is essential to ensure durability and effective waterproofing. Green roofs offer notable benefits, including reduced energy consumption and the creation of functional spaces for building occupants. However, the initial construction costs, along with ongoing maintenance and repair expenses, must be thoroughly assessed, as these expenditures may surpass the potential savings from energy conservation.

Green roofs have been used in various forms throughout history. However, the modern application of green roofs is often attributed to the Swiss architect Le Corbusier, who included them in his “Five Points of Architecture,” see, for example, Oechslin and Wang (1987). The widespread adoption of green roofs began in the 1970s in Germany, Switzerland, and France, driven by their numerous environmental and practical benefits. More recently, green roofs have gained popularity as a feature of modern urban architecture in many North American cities. This trend has been supported by municipalities and states offering incentives for the construction and maintenance of green-roofed buildings, see, for example, Berardi et al. (2014).

6.4.1: Green roof classification

6.4.2: Green roof layers

6.4.3: Green roof loads

6.4.4: Green roofs within the context of envelope inspection

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6.4.1: Green roof classification

Green roofs are typically categorized into two main types: intensive and extensive, see Jaffal et al. (2012) and Gartner (2008). The primary distinctions between these categories lie in the materials used, construction methods, and maintenance requirements.

Intensive roofs – Intensive green roofs are essentially rooftop gardens or landscaped spaces featuring diverse plant species. These systems typically require a soil depth greater than 6 to 8 inches (152 to 203 millimeters), which significantly increase the dead load on the roof structure. The deeper soil profile supports a variety of vegetation, including small shrubs and trees, but also necessitates a robust structural framework to accommodate the added weight. Additionally, intensive roofs involve higher initial construction costs and demand a comprehensive maintenance program to sustain plant health and appearance.

Extensive roofs – Extensive green roofs are less complex in design and construction compared to intensive systems. Their soil depth is generally less than 6 inches (152 millimeters), resulting in a lower dead load impact on the structural system. This makes them a more practical option for retrofit applications where green roof implementation is considered. Furthermore, extensive roofs require minimal maintenance and are more cost-effective in terms of installation and upkeep, although they typically support less diverse vegetation.

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6.4.2: Green roof layers

In an intensive green roof, generally seven layers are arranged to provide the vegetation growth, irrigation needs and water-proofing requirements – see, for example, Gartner (2008) and FMG (2007). An illustration of these layers is shown in Figure 6.8. Depending on the design, type of vegetation, building location and occupancy type, one or more of the layers shown in Figure 6.8 may not be used; two or more layers may be combined to perform multitasks.

Extensive green roofs are simpler in their layers. The vegetation is often of those requiring smaller depth to grow. In some applications, individual containers are used as both water proofing and the root barrier system.

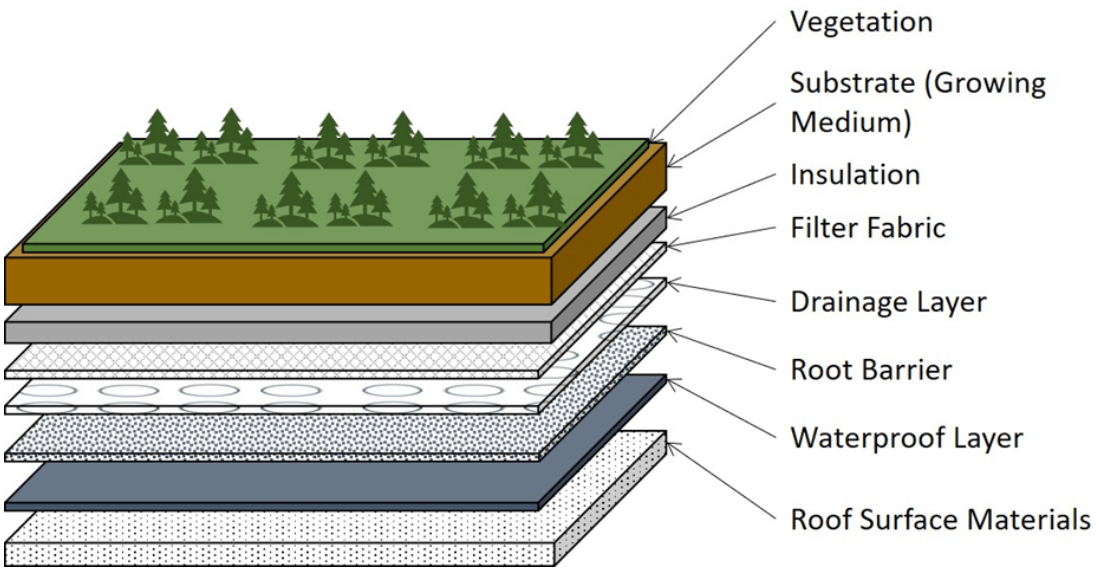


Figure 6.8 Layers making up an intensive green roof

(This figure shows details of various layers in an intensive green roof system. Seven layers are shown building over the roof surface materials. These are labeled as water proof layer, root barrier, drainage layer, filter fabric, insulation, substrate, which is the growing medium for the vegetation.)

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6.4.3: Green roof loads

Green roofs are subject to a variety of loads. Depending on the application, types of vegetation used, and usage, these loads may vary from one case to another. The following set of loads is described by Gartner (2008). An additional source of reference is ASTM (2005).

Dead loads – The dead load depends on the kind of material used to support vegetation growth and needs. Furthermore, whatever the dead load computation specifies, a 15 percent increase in the specified depth is recommended to account for future additions of growth materials. For example, if a depth of 8 inches (203 millimeters) of top soil is used, the density for this solid is about 120 pounds per cubing foot (1,922 kilograms per cubic meter). Thus, the depth to be considered will be 8 times 1.15 or 9.2 inches (234 millimeters) for dead load computation, which results in 92 pounds per square foot (4.4 kilonewtons per square meters) of additional dead load resulting from the soil alone. Additional values are considered for the vegetation, such as trees and shrubberies and water saturations as explained below.

Live loads – Live loads are determined based on the type of occupancy and local building code requirements. For example, if the intensive green roof is used as an open area for people gathering, the Chicago Building Code recommends 100 pounds per square foot of load (4.8 kilonewtons per square meter), see Section 13-52-090 of CBC (2019).

Transient live loads – According to ASTM E 2397, see ASTM (2005), a transient live load constitutes the weight of water contained in granular drainage materials and geocomposite drain layers. This load is treated as a live load and is computed based on the depth of these materials when saturated.

Snow and rain loads – These follow the recommendation of the appropriate design code, e.g., ASCE-7, see ASCE (2022).

Wind loads – Several performance requirements related to wind uplift of membranes, soil media, wind-borne debris, and building height restrictions are discussed in FM Global Data Sheet 35-1, see FMG(2007). Roofs should be designed for the envelope of wind uplift on: (a) a bare roof and (b) a saturated green roof.

Seismic loads – Current recommendations appear to indicate that the full saturated dead load of the green roof structure shall be used as part of the seismic mass. Transient live load is not included in this mass calculation.

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6.4.4: Green roofs within the context of envelope inspection

In the context of building envelope inspections, the topic of green roofs frequently arises both during initial discussions with the owner about planning the inspection and when proposing corrective measures for identified envelope distress. This discussion is not limited to cases where a green roof is already in place; it may also emerge as a suggestion from the owner to explore the feasibility of installing a green roof. Such a proposal often aims to capitalize on the numerous benefits green roofs provide, especially when repair work on other parts of the building is already under consideration.

Accordingly, this section discusses:

- Distress conditions that will need to be targeted in situations where green roofs are already in place.
- Commonly-known green roof advantages and disadvantages that need to be included as part of general information in the proposal to the owner, when they acquire about possibility of installing green roofs.

Distress conditions pertaining to green roofs – To maximize the benefits of green roofs, regular inspections, maintenance, and repair are essential. Therefore, when proposing a plan for building envelope inspection, it is crucial to obtain detailed information from the owner regarding their green roof management practices, including inspection schedules, maintenance and repair records, and details about the service provider responsible for maintaining the green roof.

The proposal should clearly define the scope of the inspector's responsibilities, especially concerning areas of the building directly or indirectly affected by the green roof's operation and management. If the owner chooses to exclude the green roof from the inspection scope and limit the contract to the wall façades, the professional of the record cannot completely disregard the green roof. This is because certain damage situations observed during the façade inspection may sometimes be linked to the green roof's operation. Such findings must be documented in the final report as areas requiring further investigations. From a liability perspective, any observed condition that raises concern must be communicated to the owner, even if it falls outside the defined scope of the contract. The professional of the record has a duty to identify and report these potential issues as part of their professional responsibility.

Leaks – The most prevalent distress condition pertaining to green roofs is leak – whether through ceiling or through parapet walls. The signs of leak appear in the form of discoloration, ceiling paster and other material spalling off, and moisture presence in the ceiling and parapet walls. Leaks generally occur because of several factors such as:

- Initial faulty design especially in installation of the waterproofing layer.
- Outgrown vegetation causing damage to the root barrier and waterproofing layer.
- Lack of periodic maintenance such as proper cleaning of debris and proper pruning and weeding.
- Deterioration of the waterproofing layer with age, and also because of the extra weight from water and soil.
- Damage from extreme loads because of strong winds, earthquakes, snowstorms, and rainstorms.
- Accidental damage during repair and/or remodeling.
- Extreme heat or cold events.
- Failure of drainage systems.

Structural damage – This damage often appears in the form of excessive deflections and cracks in the ceiling. Attributing factors include:

- Initial faulty design of roof structural members.
- Extra loads from addition of growth materials added over years.
- Damage from severe loads because of strong winds, earthquakes, snowstorms, and rainstorms.
- Changes in the occupancy and usage of the green roof and additions/remodeling.
- Failure of drainage system.

A proper line of action to address these distress conditions mainly ends up in removal of part or the entire green roof and reconstruction. This is of course will be a costly repair.

Benefits and drawbacks of using green roofs – When preparing a proposal to the owner regarding building a new green roof, benefits offered by green roofs, along with an economic study of the tradeoff between investing upfront on construction, follow-up costs for maintenance and repair, and the savings in energy consumption will need to be determined and reported.

Benefits – Among benefits offered by green roofs include:

- Adding green spaces.
- Capabilities in offering energy efficiency.
- Noise and air pollution control.
- The stormwater management capabilities by controlling runoff during heavy rains.
- Ecological preservation.

Abating heat island effect – The "heat island effect" management is a widely discussed topic in publications highlighting the benefits of green roofs in urban environments. Heat islands refer to urban areas being significantly warmer than their surrounding rural counterparts. They contribute to several negative consequences, including elevated peak energy demands during warmer months, increased air pollution, heat-related illnesses, and deteriorated water quality (USEPA, 2008).

The primary cause of heat islands is the extensive presence of dark surfaces such as rooftops, buildings, and concrete, which absorb solar heat and elevate ambient temperatures (Bianchini and Kasun, 2012). Vegetation can help mitigate this effect through evapotranspiration, which removes heat from the air, and by providing shade. Green roofs, therefore, are an effective solution for combating urban heat islands.

According to the Environmental Protection Agency (EPA), green roof surfaces can be 30 to 40 degrees Fahrenheit (about 17 to 22 degrees Celsius) cooler than conventional roofs, potentially reducing overall city temperatures by up to 5 degrees Fahrenheit (about 3 degrees Celsius). Their impact is particularly pronounced in hot and dry climates, where they have been shown to be most effective (Berardi et al., 2014).

Energy savings – Green roofs are known in reducing energy consumptions in buildings. The percentage of reduction in energy consumption depends on the types of plants and the climate. Depending on the soil depth and vegetation coverage, energy use can be reduced by up to nearly 50% compared to conventional roofs without insulation (LaRoche and Berardi, 2014). During the summer months, heat flow can be reduced by 70 to 90%, and during the winter it can be reduced by 10 to 30% (Berardi et al., 2014).

Other benefits – Reduction in air pollutants, better rainwater management and acting as a sound insulators are among other benefits of green roofs reported in literature. In particular, Bianchini, and Kasun (2012) indicate that the green roof material manufacturing process, such as polymers, with the green roof's pollution removal capacity can benefit the environment. The analysis demonstrated that green roofs are sustainable products in long-term basis. In general, air pollution due to the polymer production process can be balanced by green roofs in 13 to 32 years.

Drawbacks – Among major drawbacks of using green roofs include:

- Higher initial and future repair and maintenance cost.
- Increased load on roof structural elements.
- Discharge of hazardous materials.
- Lack of proper design in selecting an appropriate plant population that are compatible with the climate.

Initial investment and costs associated with maintenance and repair – The cost of green roof systems varies greatly based on the growing medium, roofing system, drainage system, and type/quantity of the vegetation. With the current material and labor cost (2025), the installation of a green roof is estimated to cost about 3 times more than cost of most conventional roofs. Of course, added to this cost, there are expenses associated with periodic inspections, maintenance, and repair. An economic study will then need to be conducted to determine the cost versus the return in terms of energy saving and the length of time over which the green roof would pay for itself, see also Lee (2004).

Increased load on structural members – Green roofs impose a heavy dead load and transient live load on roof structural members. As explained earlier, the materials and water saturation can easily increase the load by about 92 pounds per square foot (4.4 kilonewtons per square meter) in a typical application. The higher loads demand stronger structural members. To help reduce the weight, use of lightweight polymer materials has been considered. However, their production has shown to have negative impacts on the environment (Bianchini and Kasun, 2012).

Discharge of hazardous materials – In order to produce the polymers needed for the different layers making up a green roof, raw materials must be heated to extreme temperatures and then cooled again. This uses a great amount of energy while also releasing toxic chemicals into the environment (Bianchini and Kasun, 2012). One solution to mitigating the amount of toxins released is to use recycled polymers to create the layers.

Inadequate design and installation – Design and installation of green roofs require special expertise to make sure proper waterproofing, and drainage are maintained; and plant growth is robust. The damage to the ceiling resulting from inadequate design will end up with costly repair and reconstruction. In most cases, the installation will need to be practically flawless eliminating any potential for leak and improper drainage and plant growth.

Illustrative example – Example 6.4 in Appendix A presents an economic analysis and cost comparison between two options for a new roof, i.e., building and maintaining a green roof versus using a conventional roof system.

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6.5: GREEN WALLS

Similar to green roofs, green walls are façade systems designed to support vegetation growth on vertical surfaces. In recent years, they have gained increasing popularity due to their contributions to urban heat island mitigation, energy efficiency, and stormwater management, particularly in dense urban environments. In addition to their environmental benefits, green walls enhance the architectural character and visual appeal of buildings.

Green design strategies are increasingly incorporated into façade rehabilitation, reconstruction, and new construction projects for several reasons:

- Owner-driven interest, often influenced by exposure to existing projects that incorporate green design elements.
- Municipal requirements or incentives that encourage or mandate the inclusion of sustainable design features in both new construction and major renovations.
- Energy efficiency regulations, which frequently apply to roof and façade upgrades, making green design a viable component of compliance strategies.

Accordingly, when green façade alternatives are considered, the professional of record (i.e., the architect or engineer responsible for façade evaluation and design) is expected to be knowledgeable in the following areas:

- The fundamental principles and components of green façade and green roof systems.
- The technical and economic feasibility of such systems for a given project.
- The potential benefits to the owner, including performance, sustainability, and lifecycle considerations.
- The fire safety design considerations include fire spread risk, combustible versus non-combustible backing materials, irrigation reliability, and compliance with local building codes.

It is therefore essential that the professional of record demonstrates sufficient expertise to engage in informed and productive discussions with the owner and to incorporate appropriate provisions in the contract documents when green design alternatives are requested.

This expertise should include, but is not limited to:

- Initial construction costs and anticipated maintenance and repair costs over the system's lifespan.
- Energy performance benefits, including thermal insulation and shading effects.
- Potential damage mechanisms and distress conditions unique to green wall systems.
- Additional structural loads imposed by vegetation, growing media, and associated support systems.

Architectural and aesthetic benefits – Green walls contribute to architectural identity by softening rigid urban forms, introducing seasonal visual variation, enhancing property values when properly designed and maintained, and improving occupant satisfaction. They also strengthen the connection between people and nature within the built environment, a principle known as biophilic design. Research has shown that biophilic environments can promote health and well-being, reduce stress, enhance psychological well-being, and improve occupant productivity.

6.5.1: Kinds of green walls

6.5.2: Benefits, environmental impacts, distress conditions, and mitigations

6.5.3: Structural considerations

6.5.4: Economic considerations

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6.5.1: Kinds of green walls

In general, green walls can be categorized into three primary types: single-skin systems, double-skin systems, and living wall systems (also referred to as intermediate systems). These categories are illustrated in Figure 6.9 (see also Sharp, et al, 2008).

Single-skin systems represent the simplest and most economical option. In these systems, climbing vegetation grows directly on the building façade, with roots embedded in the ground.

Double-skin systems incorporate an independent support structure, such as a trellis, mesh, or cable network. While the vegetation roots remain in the ground, the plants grow on the support system, which may be attached to the façade or installed as a freestanding structure.

Living wall systems (intermediate systems) consist of modular panels or units installed on or integrated into the façade. These systems contain engineered growing media that support plant growth directly on the wall surface, independent of ground-based rooting.

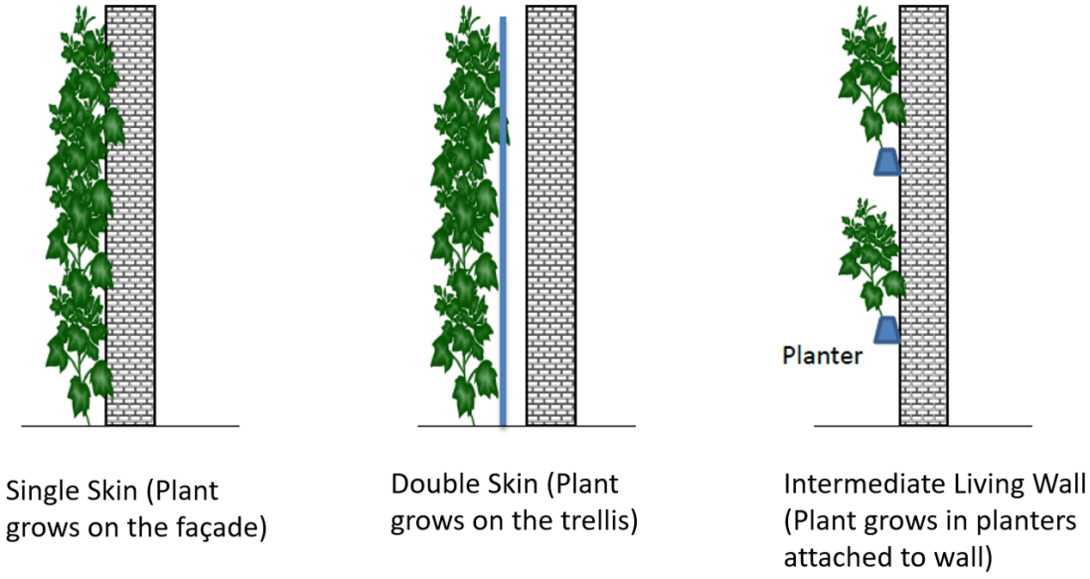


Figure 6.9 Types of green walls

(Three types of green walls are shown, the single-skin, with vegetation on the wall; the double-skin, with vegetation on a trellis and the intermediate living wall with vegetation in planters.)

Within the three categories shown in Figure 6.9, a range of terminologies is used in the literature and practice, as described below.

Green walls are often broadly referred to as living walls, bio-walls, eco-walls, or vertical gardens, indicating façade systems that are partially or fully covered with vegetation. However, it is important to distinguish between green façades and living wall systems, as these terms are not interchangeable. In general, vegetation in these systems may either be rooted in the ground and trained to grow vertically, or be contained within engineered systems that provide growing media, water, and nutrients.

A living wall system consists of pre-vegetated panels, modules, or fabric-based assemblies that are attached to a structural wall or to a freestanding frame. These modules are typically fabricated from materials such as plastics, expanded polystyrene, synthetic fabrics, or lightweight concrete. They incorporate engineered growing media and irrigation systems, enabling the support of a diverse range of plant species. Living wall systems can be installed in both tropical and temperate climates and can perform under a variety of exposure conditions, including full sun, shaded environments, and interior applications. However, they generally require significantly higher levels of maintenance compared to green façades.

In certain applications, green wall systems may also function as vegetated retaining walls. In such cases, the system is designed to stabilize soil by resisting lateral earth pressures while simultaneously supporting vegetation and mitigating erosion. These systems are commonly constructed using modular geotextile bags or interlocking units made of metal, concrete, or plastic. The modular components must provide adequate soil volume to sustain plant growth. As vegetation matures, it can substantially cover the structural elements, enhancing aesthetic value while contributing to habitat creation and ecological performance.

In contrast, green façades consist of intentionally planted vines or climbing vegetation that are rooted at the base of the structure or within elevated planters and are trained to grow along the façade surface or a supporting framework. These systems do not incorporate modular planting units. Both single-skin and double-skin configurations fall within this category. The supporting structure may be directly attached to the building façade or designed as a freestanding system.

The establishment of a green façade typically requires multiple growing seasons to achieve full coverage. The rate of growth depends on several factors, including climate, plant species, façade orientation, irrigation practices, soil depth, and nutrient availability. Green façades are commonly used to provide solar shading for glazed surfaces, reduce heat gain, and enhance the visual quality of building elements such as parking structures, walkways, and patios.

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6.5.2: Benefits, environmental impacts, distress conditions, and mitigations

Benefits – Green wall systems provide a range of environmental, functional, and economic benefits. While energy efficiency is often cited as the primary advantage, the performance benefits are multifaceted and can be summarized as follows:

- Air quality improvement: Vegetation filters airborne particulates and absorbs pollutants such as NO_x, SO_2, and CO_2.
- Urban heat island mitigation: Shading and evapotranspiration reduce ambient and surface temperatures.
- Thermal performance enhancement: Provides an additional insulating layer and reduces solar heat gain through façade shading.
- Aesthetic enhancement: Improves architectural expression and visual quality of the built environment.
- Habitat creation: Supports urban biodiversity, including birds and beneficial insects.
- Graffiti deterrence: Vegetated surfaces are less prone to vandalism.
- Energy savings: Reduces cooling loads and, in some cases, heating demand.
- Stormwater management: Intercepts rainfall, delays runoff, and reduces peak discharge.
- Noise attenuation: Vegetation and substrate can provide measurable sound absorption.
- Real estate value enhancement: Contributes to marketability and perceived building quality.
- Sustainability certification credits: May contribute to rating systems such as LEED.
- Potential thermal resistance (R-value) improvement: Particularly in systems with substrate layers.

Environmental Impacts – Green wall systems can provide measurable environmental benefits at both the building and urban scales. Modeling studies have shown that green walls can significantly reduce air pollution within urban corridors formed by closely spaced tall buildings, commonly referred to as “street canyons.” See Pugh, et al (2012).

Numerical simulations from such studies have indicated that vegetated façades can capture and absorb substantial quantities of airborne pollutants, including nitrogen dioxide (NO_2) and particulate matter (PM). The presence of vegetation alters local airflow patterns, enhances pollutant deposition on leaf surfaces, and contributes to improved air quality in densely built environments.

Green walls can also contribute to reductions in building energy consumption, provided that appropriate plant species, irrigation strategies, and system configurations are employed. Their thermal performance is influenced by moisture conditions:

- When vegetation is wet (e.g., after rainfall or irrigation), **e**vapotranspiration processes absorb heat from the surrounding air, thereby reducing ambient and surface temperatures and mitigating short-term thermal fluctuations.
- When vegetation is dry, the plant layer and associated air cavity function as an insulating barrier, reducing conductive heat transfer through the building envelope and lowering heating and cooling demands.

Further resource efficiency may be achieved by integrating water reuse systems into the green wall design. For example, greywater recycling systems can circulate partially treated wastewater through the vegetated assembly, where it is filtered by substrates such as gravel, grasses, vines, and other plant species. The treated water may then be collected in storage tanks for reuse in non-potable applications, including irrigation and, where permitted, certain household uses. Such systems must be carefully designed to comply with applicable health and environmental regulations.

Distress conditions – Despite the benefits, green walls introduce additional risks that must be considered during design, inspection, and maintenance. Distress conditions often arise from inadequate detailing, improper plant selection, or deferred maintenance.

Increased environmental and structural loads: Vegetation increases façade loading through retained moisture, snow accumulation, and wind drag. Species with dense foliage and large leaf surface areas can significantly increase dead, live, and environmental loads, potentially leading to:

- Cracking of cladding or substrate.
- Deformation of support systems (trellises, anchors, cables).
- Long-term material deterioration.

Water intrusion and moisture-related damage: Improper irrigation design, drainage deficiencies, or water retention within modules can lead to:

- Moisture infiltration into the building envelope.
- Freeze–thaw damage in cold climates.
- Corrosion of metal components.
- Biological growth (mold, algae).

Module and system failure (living walls): In modular living wall systems, excessive water, snow, or ice accumulation may result in:

- Overloading and detachment of panels.
- Degradation of growing media.
- Failure of anchorage or support systems.

Vegetation-Induced Deterioration**:** Plant selection plays a critical role in façade durability. Various kinds include:

- Climbing plants (requiring support): e.g., ivy species.
- Self-supporting plants: e.g., ferns and herbaceous species.

Certain climbers, particularly root-clinging species, may:

- Penetrate cracks, joints, and porous materials.
- Accelerate deterioration of mortar joints and coatings.
- Complicate façade inspection and maintenance.

Plants with adhesive pads (adhesive-sucker climbers), such as Virginia creeper, generally cause less structural damage but may still leave residual markings or staining upon removal.

Biological and Pest-Related Issues: Fruit-bearing or dense vegetation may attract:

- Birds, rodents, and insects.
- Nesting activity leading to localized damage.
- Organic staining and accumulation of debris.

Staining and Surface Degradation: Vegetation can cause discoloration due to:

- Organic runoff (tannins, leaf debris).
- Decaying plant matter.
- Seasonal dieback, particularly in deciduous systems.

Interference with Building Systems: Uncontrolled growth may obstruct or damage building components, including:

- HVAC equipment and air intakes.
- Electrical and communication boxes.
- Drainage systems and gutters.

Mitigation measures and maintenance requirements – To ensure long-term performance, green wall systems must be supported by rigorous inspection and maintenance programs. Recommended measures include:

- Routine vegetation management: Pruning, trimming, and species control to prevent overgrowth and maintain system balance.
- Irrigation system monitoring: Regular inspection for leaks, blockages, and proper distribution.
- Façade inspection and cleaning: Removal of dead vegetation, organic debris, and staining agents.
- Structural and anchorage inspection: Verification of the integrity of support systems, fasteners, and connections.
- Moisture management: Ensuring proper drainage, waterproofing integrity, and avoidance of water accumulation.
- Control of biological activity: Managing pests and preventing nesting where not intended by design.
- Protection of building systems: Maintaining clearances around mechanical, electrical, and plumbing components.
- Seasonal maintenance planning: Addressing freeze–thaw cycles, plant dormancy, and replanting needs where applicable.

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6.5.3: Structural considerations

The building envelope must take into account the added material of the green wall system in its structural capacity. This includes irrigation system, plants, growth media and the location of the green wall with respect to the wall, see Gartner (2008 and 2009) for additional information.

There are two primary plant support structures (see Figure 6.10):

- Modular trellis systems.
- Cable and rope wire systems.

Both systems must promote the growth of the vegetation and allow for access to water while resisting the weight of the plants during all stages of growth.

Support systems depend on the type of plants used. Climbers require vertical support to distribute the vertical load and grow upwards. Plants with branches or tree-like trunks must be anchored correctly or risk the trunks twisting around the metal frame/ support structure. The maximum height of a green wall system as one-piece system is about 75 feet (22.9 meters).

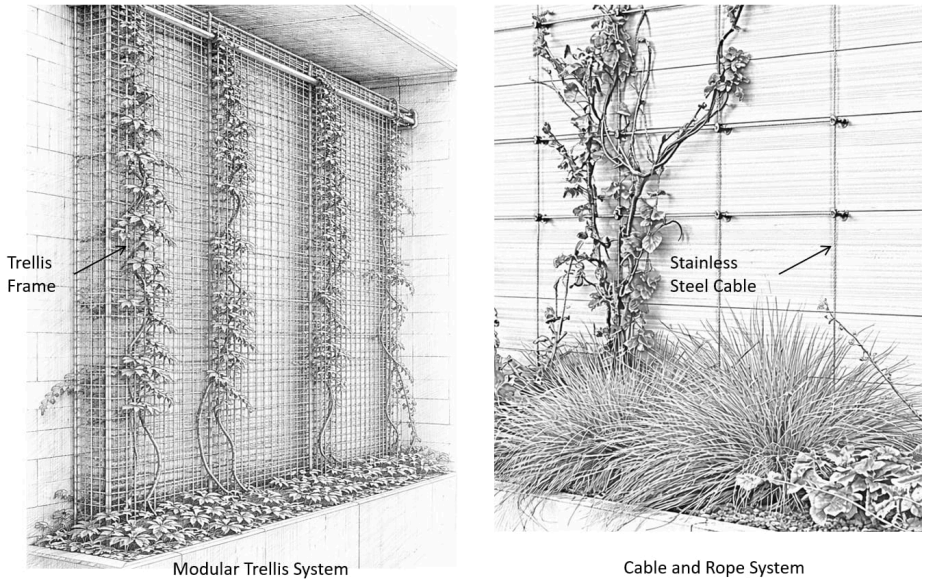


Figure 6.10 Examples of plant support systems

(Figure shows two illustrations of plant support systems, the trellis and the cable and rope system.)

The design of the building envelope must explicitly account for the additional loads and structural implications associated with green wall systems. These systems introduce dead loads from vegetation, growth media, irrigation infrastructure, and support assemblies, as well as variable loads associated with environmental exposure and plant growth. The relative position of the green wall system with respect to the primary façade (e.g., attached, offset, or freestanding) must also be considered in the structural analysis.

Modular trellis systems typically consist of lightweight yet rigid panels installed either as wall-mounted or freestanding assemblies. These panels distribute plant loads across the supporting structure and may also serve architectural functions such as shading devices or horizontal arbor elements. Material selection is critical for durability as described below.

- Wooden trellises are susceptible to moisture-related deterioration and biological decay.
- Polymeric (plastic) trellises may degrade under prolonged ultraviolet (UV) exposure and temperature fluctuations, leading to embrittlement.
- Metal trellises (e.g., stainless steel or coated steel) are generally preferred due to their superior durability, structural

performance, and resistance to environmental degradation.

Cable and wire rope systems consist of stainless steel cables, wire meshes, anchors, spacers, and connection hardware. These systems provide greater flexibility in configuration, allowing a range of geometries and patterns. Components such as cross clamps and turnbuckles are used to connect and tension the system, enabling adjustments over time as plant growth progresses. For large surface areas, rope or cable nets may be integrated to provide continuous support.

Design loads and structural effects – The structural design of green wall systems must consider several load components:

Vegetation load: The weight of vegetation varies significantly depending on species, maturity, and density. Typical values range from approximately 2 to 110 pounds per square foot (96 to 5,267 newtons per square meters). This load is in addition to the weight of growth media and irrigation systems.

Water retention and environmental loads: Vegetation can retain substantial amounts of water from rainfall, irrigation, dew, and snow accumulation. For deciduous species, environmental loading may increase the effective weight by up to two times the dry plant weight. For evergreen species, this increase may reach three times due to year-round foliage.

Self-weight and eccentricity effects: The self-weight of trellis or cable systems contributes to the overall load and may introduce eccentric loading when the system is offset from the façade. This offset can generate additional bending moments in the supporting wall and anchorage system, which must be explicitly evaluated.

Wind loads: Wind loading depends strongly on plant morphology. Species with large leaf areas or dense foliage present greater wind resistance compared to low-profile vegetation. Wind loads may also evolve over time as plant coverage increases.

Time-dependent load increase: Green wall systems are dynamic, living systems. As vegetation matures, both dead and environmental loads increase. Structural design must account for future growth conditions, not only initial installation loads.

Proper maintenance – including pruning, irrigation control, and periodic inspection – is essential to prevent unintended load increases and to ensure long-term structural performance.

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6.5.4: Economic considerations

The cost of green wall systems varies widely depending on multiple project-specific factors, including:

- Coverage area, which influences initial construction, irrigation infrastructure, and long-term maintenance costs.
- Plant selection, including species, density, and growth characteristics.
- Climate and geographic location, affecting plant viability, irrigation demand, and durability requirements.

Careful consideration of site conditions is essential during design. Key factors include:

- Use of native species, which typically improves survivability and reduces maintenance requirements.
- Avoidance of invasive species, which may damage adjacent vegetation and the supporting structure.
- Water demand, which directly impacts irrigation system design and operational costs.

The successful implementation of a green wall system generally requires specialized expertise, including horticultural consultation, irrigation design, and façade engineering. These services contribute to overall project costs but are essential for long-term performance.

Green wall systems can add approximately 15% to 20% to the total cost of façade rehabilitation or reconstruction, depending on system complexity. Additionally, expert analysis may be required to quantify potential energy performance benefits, such as reductions in heating and cooling loads.

Lifecycle costs must also be considered. Vegetation may require periodic replacement or replanting, depending on species and environmental conditions, leading to recurring expenses.

Typical cost ranges (in 2026 dollars) are as follows:

- Climbing plant systems (cables, mesh, or trellis): approximately $50 to $75 per square foot.
- Living wall systems (modular panels or integrated systems): approximately $80 to $200+ per square foot, depending on system complexity and technological integration.

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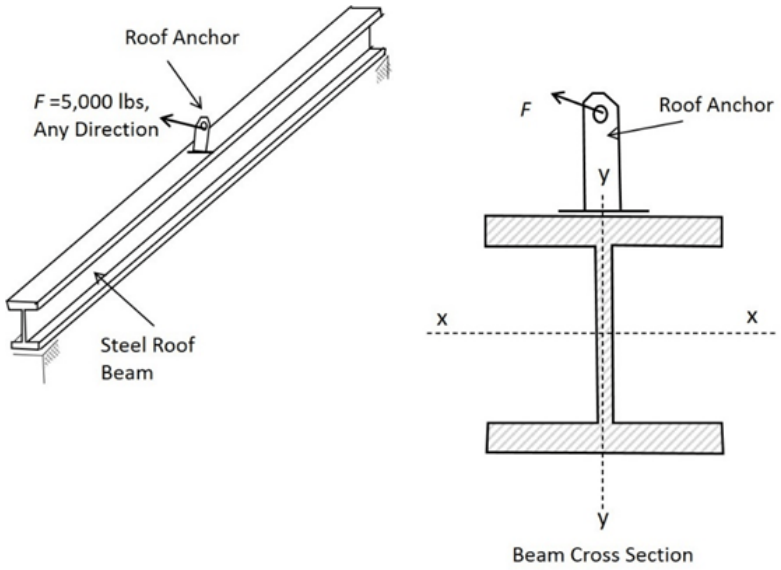
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6.7: Exercises

E6.1 In a façade inspection planned for a building, roof anchors that support a moveable scaffold must be welded to steel roof beams as shown. As you recall, such anchors are designed for 5,000 pounds (22.2 kilonewtons) of force applied in any direction. In this building, each beam will need to support one anchor at its midspan. A preliminary investigation and review of structural drawings reveal that beams are steel W-sections and each will need to support the entire 5,000-pound (22.2-kilonewton) load without reliance on bracing and lateral supports (see the accompanying figure). Considering this beam with the loading shown, which of the following situations is most critical for the beam analysis?



(This figure shows an I-beam on the left with a bracket installed on the top flange with the 5,000-pound load applied on it. To the right, a cross-section details is shown with the bracket and the load.)

a) A bending moment with respect to the x-x axis of the cross-section is most critical.

b) A bending moment with respect to x-x, and a bending moment with respect to y-y axis plus a torsional moment with respect to the longitudinal axis of the beam will be most critical.

c) The shear force along y-y axis together with a bending about x-x axis is the most critical one.

d) The axial load along the length of the beam together with a torsional moment about the x-x axis is the most critical one.

E6.2 A parapet wall has a height of 4 feet (1.22 meters) and a width of 2.5 feet (0.76 meters). It is part of the façade in a building with a roof of 60 feet (18.3 meters) in length. The ground snow load for the area is estimated as 30 pounds per square foot (1.44 kilonewtons per square meter); and the building is designated as Category II. The roof is flat.

a) Determine the length (w) of snow drift.

b) Determine the maximum snow load on the roof at the location of the parapet wall using pounds per square foot.

E6.3 What are EPDM and TPO?

a) Types of materials used as flashing in parapet walls.

b) Special synthetic membranes used as roofing materials.

c) Types of caulking materials.

d) Plastic sheets used as tarps.

E6.4 Which of the following statements is least accurate?

a) A good practice in parapet wall detailing requires appropriate flashing on the roof side as well as below coping stone.

b) Coping stones without “drip groves” are considered deficient.

c) The wind load is a major loading condition affecting parapet walls.

d) In general parapet walls may be ignored as part of façade inspection because it is more a roof component.

E6.5 In inspecting a façade, two diagonal cracks were observed at around 45-degree angles above a large window in the parapet wall. Discuss potential causes and lines of action to address this distress condition.

E6.6 A parapet wall has an effective thickness of about 14.25 inches (362 millimeters) and a height of 3.5 feet (1.07 meters) above the roof level. Compute the maximum stress in the mortar joint for a wind pressure of 30 pounds per square foot (1.44 kilonewtons per square meter). The wall density is 140 pounds per square foot (683.5 kilograms per square meter) of the wall surface.

E6.7 Which of the following is most probable cause of coping stone damage in brick parapet walls?

a) Fire protection violation.

b) Low quality brick units.

c) Weather; water washout; periodic freeze/thaw cycles; poor workmanship; age.

d) Inadequate wall height.

E6.8 Which of the following is not likely to contribute to crack formation in parapet walls?

a) Persistent wet/dry cycles.

b) Lack of expansion joints, inadequate lintels, thermal expansion and contraction.

c) Poor flashing detail on the roof side.

d) Inadequate height.

E6.9 An intensive green roof has a depth of 6 inches (152 millimeters) for plant growth. The material used is compacted top soil with a density of 100 pounds per cubic foot (1,602 kilograms per cubic meter). The weight of other layers amount to be 7.5 pounds per square foot (36.6 kilograms per square meter). Compute the extra load on the roof because of the green roof application when soil is fully saturated. Note About 15% increase is considered for the growing medium height for future soil addition.

E6.10 Which of the following is not considered among the main benefits of a green roof?

a) Savings in energy consumption.

b) Ecological preservation.

c) Low maintenance cost.

d) Better rainwater management.

E6.11 An owner is evaluating the benefits of building a green roof for a new residential complex versus a conventional roof system. The following is the cost information for an intensive green roof (Option 1) and a conventional roof (Option 2)

Option 1: Intensive Green Roof

Total initial cost: $166,000

Annual repair and maintenance: $6,000 in the first year. This will be increased $500 every year thereafter.

Saving in energy consumption for the entire complex: $6,000 per year

A one-time incentive by the local government paid to the owner in year 5 is $4,000.

Option 2: Conventional Roof

Initial cost: $45,000

Annual repair and maintenance $2,000 per year. This will increase $400 every year thereafter.

No additional cost

Compare these two options for a service life of n = 20 year and an interest rate of i = 0.06 per year using an engineering economics analysis (see Chapter 9).

E6.12 What is the primary cause of the heat island effect in urban areas?

a) The presence of too many trees and vegetation.

b) Extensive use of dark surfaces that absorb solar heat.

c) Excessive wind circulation in city environments.

d) Low levels of energy consumption in buildings.

E6.13 How do green roofs help mitigate the heat island effect?

a) By reflecting more solar radiation than conventional roofs.

b) By increasing the amount of artificial cooling systems in buildings.

c) Through evapotranspiration and providing shade.

d) By reducing the number of buildings in urban areas.

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7: ANCILLARY WALL COMPONENTS, SHELF ANGLES, LINTELS, AND GIRT BEAMS AND THEIR ATTACHMENT TO SUPPORT STRUCTURE

Chapter Synopsis

This chapter covers proper design for several wall components including shelf angles, lintels, and girts. Shelf angles carry the weight of veneer walls, while lintels are beams spanning over openings in walls and carry the weight of walls above them. Girts are horizontal beams used in façades to carry and distribute the wind loads to the structural members of the building. Furthermore, connections of these elements to the support structure (whether in the form of anchors, bolts or welds) are covered. Illustrative examples pertinent to materials presented in this chapter are offered in Appendix A.

7.1: Introductory Remarks

7.2: Shelf Angle Detail and Design

7.2.1: Sizing the Angle

7.2.2: Structural design of shelf angles

7.3: Shelf Angle Connection to the Structural System

7.3.1: Shelf angle connection to concrete using anchors

7.3.2: Shelf angle connection to masonry using anchors

7.3.3: Shelf angle connection to structural steel

7.4: Lintel Design

7.5: Shelf Angle and Lintel Damage Issues

7.6: Girt Design and Damage Issues

7.7: Chapter References

7.8: Exercises

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7.1: Introductory Remarks

Condition assessment of walls also involves a close examination of all ancillary components that contribute to the architectural aesthetic and structural integrity of the building envelopes. Several such components act as flexural structural members in the form of beams. And as such, they will need to be properly designed to safely carry their loads and provide the stability for the façades. These elements often are among the first wall components that undergo damage, usually in the form of corrosion, which stems from improper architectural and structural design.

Figure 7.1 shows a lintel damage in the form of corrosion that can be attributed to lack, or improper design, of a proper moisture control system including flashing, weep holes, and wicks.



Figure 7.1 Photo shows corrosion damage to a lintel (photo courtesy of Dearborn Engineering Corporation)

(This figure shows a beam spanning above an entry door exposed to reveal its condition. The beam is a steel structural member with signs of corrosion.)

In cases where damage to ancillary façade components is suspected during an inspection, it is often necessary to remove sufficient amounts of cladding, brick and CMU blocks, and other covering materials to expose these elements for detailed examination. This allows for a proper assessment of both the cause and extent of the deterioration. Regardless of the specific cause, remedial actions must ensure that the affected component achieves both adequate architectural performance and structural integrity.

From an architectural standpoint, proper design focuses on details that prevent moisture intrusion and deterioration. Key considerations include the correct installation of flashing, provision of weep holes, and other drainage and ventilation details discussed in Chapter 2. These measures are critical to managing water infiltration and prolonging the service life of façade systems.

From a structural perspective, design involves accurate determination of applied loads, appropriate sizing of members, and verification of performance under conditions of flexure, shear, and deflection. Shelf angles and lintels, for example, must be designed with two primary objectives:

- Preventing future deterioration due to corrosion of the embedded or exposed steel elements.
- Providing sufficient strength and stiffness to avoid distress in the supported wall, such as cracking or displacement.

For girts, the principal design concern is the management of wind-induced forces acting on the façade. Proper girt design ensures that lateral pressures are safely resisted and effectively distributed to the primary structural system. This is especially critical in

cases where large portions of a façade are exposed to significant wind loads, such as in tall buildings or those located in high-wind regions.

Accordingly, this chapter addresses the design of shelf angles, lintels, and girts from both structural and architectural perspectives. Emphasis is placed on achieving the required load-carrying capacity while also integrating effective strategies for moisture management and corrosion prevention, thereby ensuring long-term durability and performance of the building envelope.

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7.2: SHELF ANGLE DETAIL AND DESIGN

Shelf angles (also referred to as steel relief angles) are attached to the structural systems and are designed and detailed properly to provide support for veneer walls. Usually, they are installed at every other floor level. Figure 7.2 illustrates an example of a veneer wall supported by a shelf angle. Notice that in this particular case, the structural system, providing support, is a reinforced concrete spandrel beam. This is a beam that is constructed around the edge of each floor and is integrated with the floor slab.

Shelf angle details, as shown in illustrations in this section, as well as the design procedure and examples provided in the Appendix are adopted from Mohammadi (2021).

As seen in Figures 7.2, the shelf angle detail also displays the location of the flashing. In addition, anchor bolts are shown, which attach the shelf angle to the structural system. Also notice that a sealant material and an element called “backer rod” are used below the outstanding leg of the angle. A backer rod is a rope-like flexible material with a round cross section that is available in a variety of thicknesses. The backer rod, along with the sealant compound, is used to prevent the moisture from entering the cavity behind the wall. Ordinarily, a mortar trap is also used - however, in this figure, it is not shown for clarity. A similar detail with mortar trap is presented in Figure 2.5.

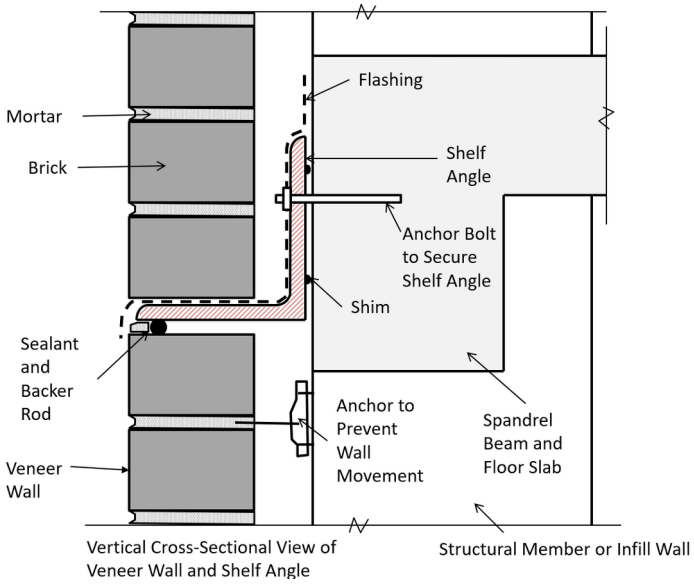


Figure 7.2 Shelf angle to support veneer wall (adopted from Mohammadi, 2021)

(This figure is an illustration of a shelf angle detail. The cross-section of the shelf angle and the veneer wall it supports are shown in a vertical cross-section detail of the wall, and the supporting concrete structural members. Anchors connecting the angle to the structural members are also shown.)

Shelf angles are designed for the dead load of the veneer walls they support. In designing them, we must consider

- Sizing the angle for proper support of the wall.
- Designing the angle for proper thickness.
- Designing for proper attachment of the angle to the structural system.

When the structural system is a masonry wall or a reinforced concrete frame, the attachment of the angle to the structure is achieved by using anchor bolts (as shown in Figure 7.2). A proper design of the anchor bolts requires selecting suitable values for the cross-sectional diameter, embedment length, and the spacing between anchors. The recommendations of the American Concrete Institute (see, for example, ACI, 2014) are used for this purpose. In situations where the structural system is a steel frame, the angle attachment is done using welds or bolts. The design in this case requires selecting suitable values for the weld size and weld length based on a electrode type. Bolt design requires selecting a proper cross-sectional diameter for the type of bolt used (A325 or A490) following the bolted connection design provisions of the American Institute of Steel Construction (see,

for example, AISC, 2017).

7.2.1: Sizing the Angle

7.2.2: Structural design of shelf angles

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7.2.1: Sizing the Angle

The angle must have an adequate width to safely support the wall. Figure 7.3 shows the longitudinal and cross-sectional detail of a shelf angle. As seen in this figure, the weight of the wall is applied as a uniformly distributed load. As a conservative design practice, the load W from the wall is applied at the tip of the angle (see Figure 7.3). However, as indicated by McGinley (2013), this condition never happens; and as such, it is reasonable to take the point of application of the veneer load at the center of the wall. This is depicted in Figure 7.4, where the modular dimensions of a brick unit (with consideration for the finished size with mortar in place) are shown as used in a 4-inch (102-millimeter) wall.

As an example, an angle with 4 inches (102 millimeters) of outstanding leg (horizontal leg) is used to support the wall. This is the shortest angle width that can be used to accommodate the wall, since we must have at least 75% of the width of brick wall supported. For a 4-inch (102- millimeter) wall, the effective width of the wall is 4 minus 3-8th of an inch, i.e., 3.625 inches (92 millimeters). If an angle with 4 inches (102 millimeters) of outstanding leg is used, such as a 4 by 3.5-inch angle with a quarter of an inch thickness, the flat area of the angle leg is 4 minus 5-8th of an inch, i.e., 3.375 inches (86 millimeters). The 75% of the wall thickness is 0.75 times 3.625 or 2.72 inches (69 millimeters). Therefore, the angle size is acceptable. However, this condition provides only a very small gap area between the veneer wall and the inner face of the vertical leg of the angle (about 1 inch or 25 millimeters). In practice, this gap is wider and is about 2 inches (51 millimeters). In this situation, certainly a larger angle size (for example, an angle with a 6 inches of horizontal leg) will be needed.

For the dimensions shown in Figure 7.4 and an angle thickness of quarter of an inch (6 millimeters), distance e, representing the location of the applied load, can be computed considering that the effective width for a 4-inch (102-millimeters) wall is 3.625 inches (92 millimeters).

e =3.625/2+1+0.5x(1/4) = 2.94 inches (or 75 millimeters)

This is measured from the center of the thickness of the vertical leg of the angle. The 5-8th of an inch (16 millimeters) dimension we used in Figure 7.4 depends on the angle thickness. Information on this dimension for angles of different sizes and thicknesses can be found in AISC (2017). In general, if a cavity gap equal to c is used, for an angle thickness equal to t, and the effective wall thickness of T_E, distance e is

$$e = \frac{T_E}{2} + c + \frac{t}{2} \tag{7.2.1.1}$$

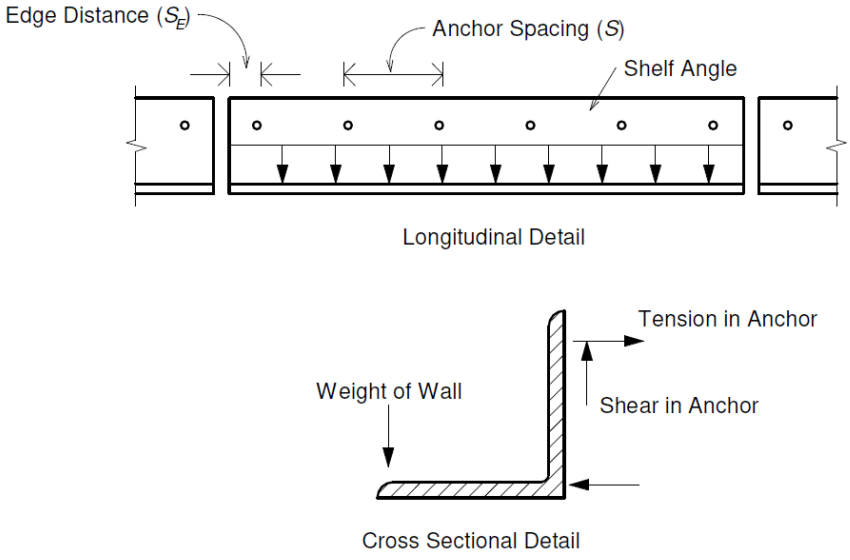


Figure 7.3 Shelf angle structural details in longitudinal and transverse directions

(This figure shows the longitudinal view of a shelf angle with its linear load. The cross-section of the angle with the free-body diagram of the applied loads and the reactions are also shown. The reactions are a tensile and shear force at the location

of the connecting anchor to the wall and a compressive force at the location of the outstanding leg of the angle.)

Obviously, with a wider cavity gap, the size of the angle will need to increase. For example, with c = 2 inches (51 millimeters) and an angle thickness of three quarters of an inch (19 millimeters), the distance e is computed as

c= 3.625/2 + 2 + 0.5x(3/4) = 4.19 inches (106 millimeters)

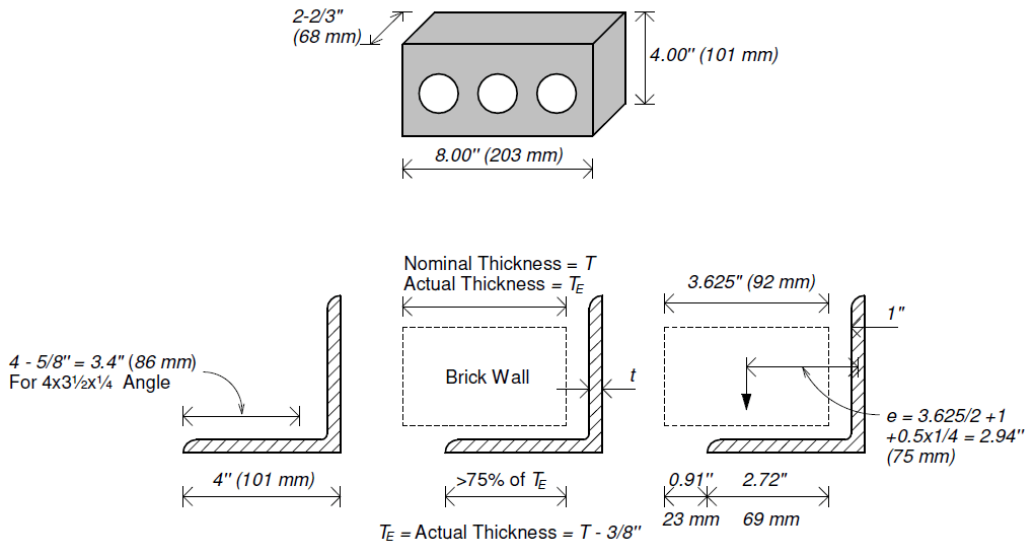


Figure 7.4 Angle size for a 4-inch (102-millimeter) wall – note that the wall needs to be supported over at least 75% of its width with the tip of the angle hidden

(This figure shows details of the typical dimensions of a brick unit used in 4-inch (102-millimeter) veneer walls. Three angle cross-sections are also shown revealing how to find the distance e, which is the distance of the location of the applied load from the weight of the wall to the center of the vertical leg of the angle. The angle used is 4 by 3 inches and a quarter of an inch thickness. The dimension e is computed to be 2.94 inches (75 millimeters) for 1 inch (25 millimeters) of gap between the wall and the vertical leg of the angle.)

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7.2.2: Structural design of shelf angles

For a shelf angle that is anchored to a masonry wall or to a reinforced concrete structural system, the structural design of the angle is performed for

- The angle thickness to determine whether it is adequate.
- The size and embedment length needed for anchor bolts.
- The anchor spacing.

In practice, the design starts by selecting the spacing first, since the spacing controls and determines the appropriate height for the veneer to support itself (between the anchors) and the effective width of the angle for the calculation of the stresses in the angle. Figure 7.5 illustrates the angle and its structural design parameters.

The calculation of the load is done for a length equal to the anchor spacing (S); however, calculation of all cross-sectional properties is conducted for an effective angle length of b. The effective length b is determined based on the nominal wall thickness (T) and the anchor spacing S using the following relationship.

$$b = 4T \leq S \tag{7.2.2.1}$$

It is noted that even though we have referred to b as effective length, in reality and as shown in Figure 7.5, when computing the cross-section properties of the outstanding leg of the angle, which supports the load W, this dimension is considered the width of a rectangular section (b with a thickness of t that resists the effect of the load W.

Based on the anchor spacing, and using the masonry tensile capacity in bending, we can determine the relation between the height of the masonry and the anchor spacing and decide whether the masonry is able to support itself over the distance S. The load from the wall is a linear load assumed uniformly distributed (see Figures 7.3 and 7.5). Considering the wall spanning over a distance S between two adjacent anchors, the uniform load (q) in pounds per linear foot (kilonewtons per meter) from the weight of the wall and the bending moment (M) in foot-pounds (meter-kilonewtons) developed are, respectively

$$q = wh \tag{7.2.2.2}$$

$$M = \frac{qS^2}{8} = \frac{whS^2}{8} \tag{7.2.2.3}$$

In which w is the wall density (in units of pounds per square foot or kilograms per square meter) and h is the wall height. The wall vertical cross-section resists this moment. The section modulus (S_x) of a wall with a nominal thickness of T and the effective thickness T_E is

$$S_x = \frac{T_E h^2}{6} \tag{7.2.2.4}$$

Where, the effective thickness is T minus 0.375 inches (9.5 mm).

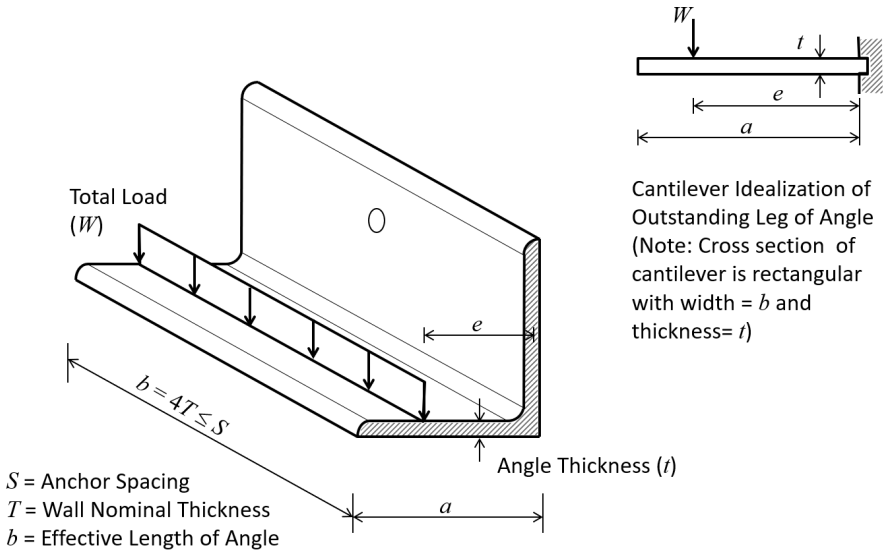


Figure 7.5 Structural design parameters of shelf angle

(This figure shows the idealization of the outstanding leg of the angle as a cantilever. An isometric detail of the angle is also shown with effective width b identified, which is used for computing the load W on the angle.)

The bending stress at the vertical cross-section of the veneer wall (affecting the mortar) is

$$f_b = \frac{M}{S_x} = \frac{6wS^2}{8T_E h} \tag{7.2.2.5}$$

Denoting the allowable bending stress for the masonry (parallel to the bed joint) as F_b, the design is then described through the expression $f_b \leq F_b$. Thus

$$\frac{6wS^2}{8T_E h} \leq F_b \tag{7.2.2.6}$$

Solving Equation 7.2.2.5 for S results in the following equation for the maximum limit for the anchor spacing to allow for the veneer wall to support itself.

$$S \leq \sqrt{\frac{8T_E h F_b}{6w}} \tag{7.2.2.7}$$

As a conservative value, McGinley (2013) suggests F_b = 15 pounds per square inch (103 kilopascals) for Type N mortar, although higher values are considered to be more reasonable.

As seen in Figure 7.5, a portion of the wall will cantilever itself where the last anchor is located. Denoting the distance between the center of the last anchor to the edge of the angle (edge distance) as S_E, the wall moment, M, and the bending stress, f_b, are

$$M = \frac{qS_E^2}{2} = \frac{whS_E^2}{2} \tag{7.2.2.8}$$

$$f_b = \frac{M}{S_x} = \frac{3wS_E^2}{T_E h} \leq F_b \tag{7.2.2.9}$$

Using Equation 7.2.2.9, the maximum edge distance is

$$S_E \leq \sqrt{\frac{T_E h F_b}{3w}} \tag{7.2.2.10}$$

For determining an appropriate angle thickness t to support the weight of the wall, the outstanding leg of the angle is treated as

a cantilever system (as illustrated in Figure 7.5). The cross-sectional properties of the cantilever shown in Figure 7.5 are obtained from the following equations:

Cross-sectional area: $A = b \times t$

Moment of inertia: $I = \frac{b \times t^3}{12}$

Section modulus: $S_x = \frac{b \times t^2}{6}$

Basic equations needed for the computation of internal forces and stresses are:

Bending moment: $M = W \times e$

Shear: $V = M$

Deflection at the tip of the angle: $y = \frac{W \times e^2}{6EI}(3a - e)$

Bending stress: $f_b = \frac{M}{S_x}$

Shear stress: $f_v = \frac{3V}{2A}$

The design is generally governed by flexure. Once an appropriate thickness for the angle is selected, it is then checked for shear and deflection. Using the load and resistance factor design (LRFD) method, the design equation is represented by

$$M_u \leq \phi M_n \tag{7.2.2.11}$$

In which, M_u is the factored applied moment, M_n the nominal bending capacity, and ϕ the capacity reduction factor (which is considered as 0.9 for flexure in this case). Since the weight from the wall is all dead load, using a load factor equal to 1.4, the factored applied moment is

$$M_u = 1.4W \times e \tag{7.2.2.12}$$

in which,

$$W = (w \times h + q_a) \times S \tag{7.2.2.13}$$

Where w is weight from the wall density (using pounds per square foot or kilonewtons per square meter) and q_a is the load from the weight of the horizontal leg of the angle (using pounds per foot or kilonewtons per meter) with the unit of W in pounds or kilonewtons. The nominal bending capacity is

$$M_n = F_y \times S_x \tag{7.2.2.14}$$

In Equation 7.2.2.14, we are using the yield capacity (F_y) as the limit state for the strength of the material and the elastic section modulus S_x (rather than the plastic modulus Z_x), which indicates our assumed limit state is the onset of plastic hinge formation, with the section yielding at the extreme upper and lower fibers of the cross section, rather than the full plastic hinge formation. Using Equations 7.2.2.11 to 7.2.2.14 and substituting for the section modulus S_x, an equation for the angle thickness can be written as follow.

$$t \geq \sqrt{\frac{8.4(w \times h + q_a)S \times e}{\phi F_y b}} \tag{7.2.2.15}$$

It is noted that by equilibrium, the vertical leg of the angle will also be under the applied moment M_u; however, in addition to the bending stress, this leg of the angle is also subject to a tension stress as follows:

$$f_t = \frac{1.4W}{b \times t} = \frac{1.4(w \times h + q_a)S}{b \times t} \tag{7.2.2.16}$$

This stress value needs to be checked; but usually it does not add any appreciable value to the bending stress.

Illustrative Examples – Examples 7.1 and 7.2 in Appendix A demonstrate the applications of materials and equations presented in this section for shelf angle design.

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7.3: SHELF ANGLE CONNECTION TO THE STRUCTURAL SYSTEM

Shelf angles are connected to the structural members via fasteners (bolts), anchors, or welds. If the structural system supporting the angle is reinforced masonry or concrete, the connection is achieved by using anchors. In cases where the structure is a steel system, bolts or welds are used. When the main framing system of the building is made up of wood, shelf angles are connected to the structural system via bolts with or without steel brackets.

7.3.1: Shelf angle connection to concrete using anchors

7.3.2: Shelf angle connection to masonry using anchors

7.3.3: Shelf angle connection to structural steel

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7.3.1: Shelf angle connection to concrete using anchors

Anchor forces and design parameters – Anchors are used to connect shelf angles to reinforced concrete structural elements such as slabs, spandrel beams, or other types of structural members. Anchors are a type of fastener embedded into the structural member and are designed mainly for resistance against a combination of a pull-out (tension) and a shear force.

The applied moment $M = W \times e$ is balanced by a tension force in the anchor and a compression as shown in Figure 7.6. Note that a more accurate value for the moment is $M = (W \times e + q_a \times S \times \frac{a}{2})$, where q_a is one half of the weight of the angle in unit length, S is the anchor spacing, and a is the width of the outstanding leg of the angle (for example, for a 6 by 6-inch angle, a = 6 inches or 152 millimeters). Conservatively, and as an approximation, we can simply take the moment as $M = (W + q_a \times S) \times e$ or if we ignore q_a, the moment is simply $W \times e$. In any case, the tension in the anchor will be $T_a = M/y$ and shear $V_a = W + 2q_a \times S$.

The main parameters used for the design of anchors are the embedment length h_{ef} and the anchor diameter d_a, which is the outside diameter of the anchor or the shaft diameter in headed anchors and studs. There are two types of anchors:

- Conventional types made up of threaded or hooked bars.
- Commercial anchors, which are patented and are distributed by various manufacturers (such as Hilti® and Strong-Tie®).

Commercial anchors are especially becoming popular because of their variety in terms of strength and versatility, and the services provided by the manufacturers through application software and expert consulting. Regardless of the type used, we must understand the modes of failure of the anchor and compute its capacity accordingly.

Strength computation – For strength computation of anchors, the provisions of the 2014 edition of ACI 318 are used in this book. The reader is referred to the latest editions of the ACI 318 for any revisions implemented in the strength computation procedure. Chapter 17 of ACI (2014) covers the design requirements for anchors under tension, shear, or a combination of these two kinds of forces. The materials presented herein outline the strength design method. The strength design method is the term used by the ACI and is the equivalent of the LRFD method. Therefore, all applied loads will need to consider appropriate load factors and a combination of different kinds of loads as stipulated in Section 5.3 of ACI (2014). For example, if only the dead load is involved, the load factor is 1.4. While if both the dead and live loads are present, the load factors are 1.2 and 1.6 for the dead and live loads, respectively. The resistance reduction factor varies depending on the mode of failure as explained later.

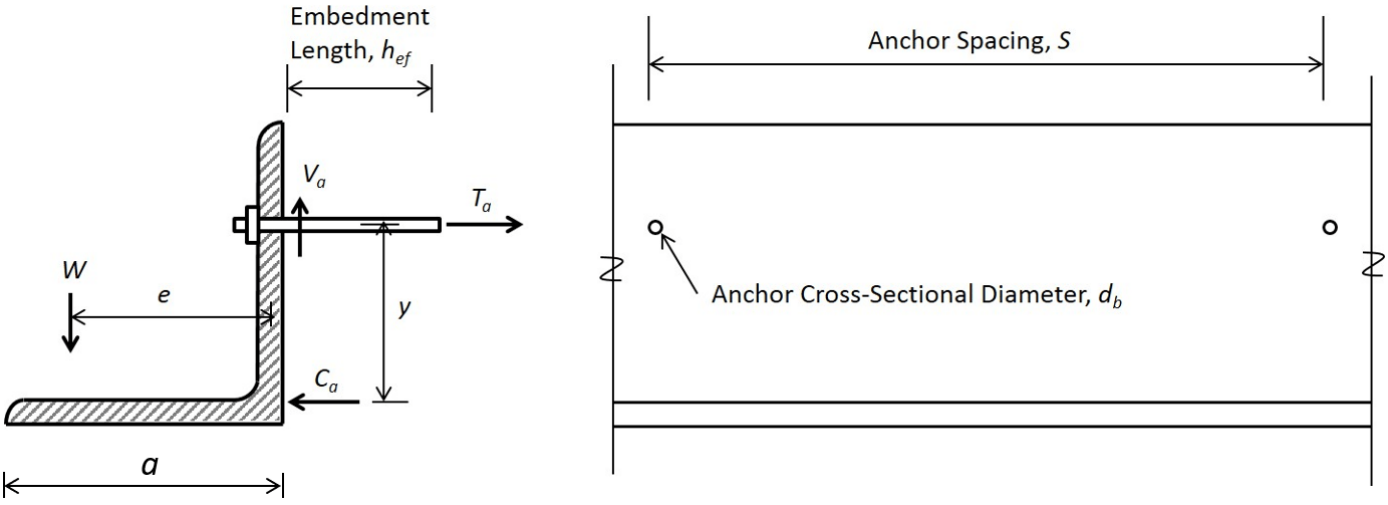


Figure 7.6 Anchor forces and design parameters

(This figure shows a free-body diagram of forces on the anchors connecting the shelf angle to the concrete or masonry structural system. A tensile and a shear force are shown for the anchor.)

The ACI provisions apply to cast-in, post-installed expansion (torque-controlled and displacement-controlled), undercut, and adhesive anchors (Mohammadi, 2021). In addition to Chapter 17 of ACI (2014), the reader is referred to two additional documents

pertinent to testing and assessment criteria for mechanical and adhesive anchors. These are ACI 355.2 (2007) and ACI 355.4 (2011). ACI (2014) permits using the strength design of anchors based either on computation using design models that satisfy the requirements of Section 17.3.2 of ACI 318, or on test evaluation using the 5 percent fractile of applicable test results. The 5 percent fractile concept is used to come up with a nominal strength value (R_n), with 95% of the samples having a strength equal to or larger than this nominal value. Using a normal probability density function for the anchor strength, and knowing that the value corresponding to 0.95 probability is 1.65 (see, for example, Khisty, Mohammadi and Amekudzi, 2012) the design strength value $R_n = R_m - 1.65 \times S_D$, in which R_m is the mean and S_D is the standard deviation of data from the test samples used in determining the strength.

Strength calculations and tests for tension and shear account for six failure modes in tension and three in shear. Depending on the failure mode, we may need to consider either individual anchors or groups of anchors acting together to resist failure. When anchors are spaced relatively far apart, individual anchor failure typically governs. The strength values corresponding to the six modes of failure in tension are

(a) Steel strength of anchor in tension.

(b) Concrete breakout strength of anchor in tension.

(c) Concrete side-face blowout strength of headed anchor in tension.

(d) Concrete splitting strength.

(e) Pullout strength.

(f) Bond strength of adhesive anchor in tension.

As indicated, each mode is considered individually, with a sole anchor participating in resisting the applied forces. However, in certain situations, the anchor resistance needs to consider a group of anchors as well, since the resistance is affected with adjacent anchors. For example, when considering the concrete breakout mode of failure, a group of anchors will all participate in strength computation when anchors are very close to one another. Thus, in this situation, we need to look at the strength of any one anchor as well as the group strength and select the value that is more conservative for the anchor strength. The ACI (2014) provisions outline the situations when group effect will need to be considered.

In the case of shear, the strength calculations are for

(a) Steel strength of anchor in shear.

(b) Concrete breakout strength of anchor in shear.

(c) Concrete pryout strength of anchor in shear.

The modes of failure of anchors in tension and shear are depicted in Figures 7.7 and 7.8. As indicated in Chapter 17 of ACI (2014), depending on anchor spacing and edge distance, these modes can affect individual anchors or anchors in groups. The decision of whether the strength computation will need to only consider a single anchor, an individual anchor in a group, or a group of anchors depends on the mode of failure, the anchor spacing, and the edge distance.

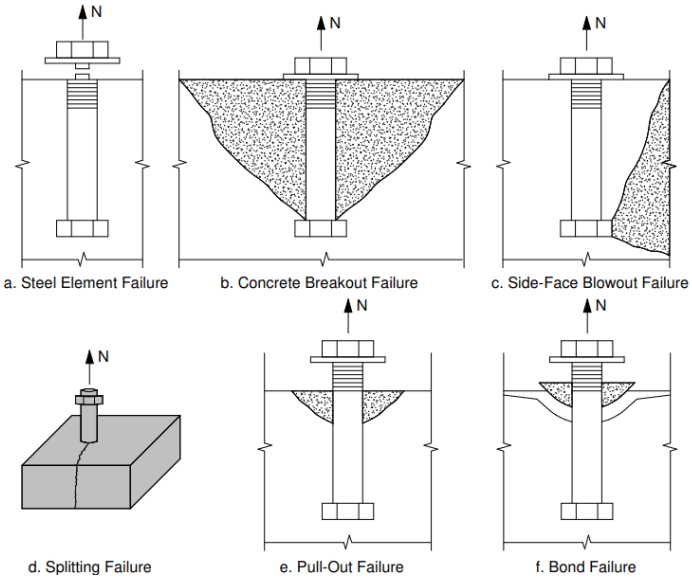


Figure 7.7 Anchor tension modes of failure, adopted from ACI (2014)

(Details of six tension modes of failure are shown designated by letters a through f. Detail a shows an anchor with its head separated because of tension. Detail b shows a conical potion of concrete separated. Detail c shows a portion of concrete at the edge separated. Detail d shows a concrete split. Detail e shows the anchor pulled out. Detail f shows loss of bond between concrete and stem of the anchor.)

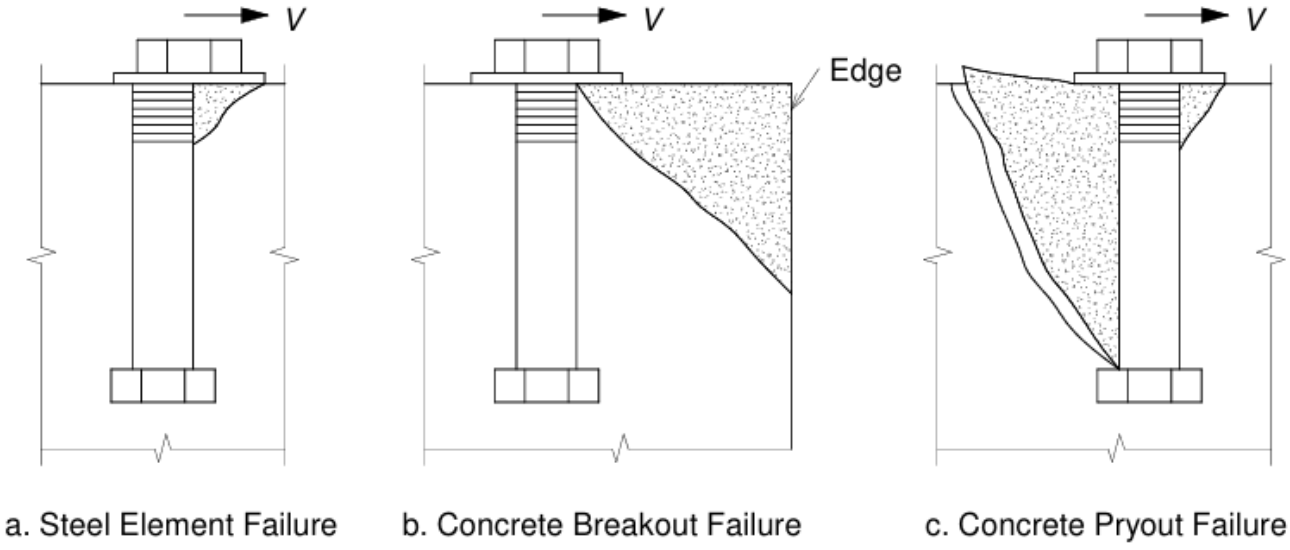


Figure 7.8 Anchor shear failure modes, adopted from ACI (2014)

(Three details for shear failure in anchors are shown. Detail a shows shear failure in the anchor metal. Detail b shows a portion of concrete is separated at the edge. Detail c shows a portion of concrete is separated behind the anchor.)

Anchor group effects will need to be considered for two or more anchors for the following three modes of failure and anchor spacing conditions listed.

- For concrete breakout in tension, when anchor spacing is less than $3h_{ef}$.
- For bond strength in tension, when anchor spacing is less than $2c_{Na}$.
- For concrete breakout in shear, when anchor spacing is less than $3c_{a1}$.

In which

h_{ef} is the effective embedment length of the anchor.

c_{Na} is the projected distance from center of an anchor shaft (on one side of the anchor) to the edge of concrete needed for a full development of the bond strength of a single adhesive anchor.

c_{a1} is the distance from the center of an anchor shaft to the edge of concrete in one direction.

In cases where the force is in shear, c_{a1} is measured in the direction of shear. However, when the force is in tension, the distance is the minimum edge distance. It is noted that some strength equations for certain modes of failure also refer to a distance c_{a2} . This is the edge distance perpendicular to c_{a1}, and at times also indicates the largest edge distance. It is emphasized that in shelf angle design, it is uncommon to have anchors that are too close to the each other that may require consideration for the group effect. In most applications, single anchor effects and a handful of failure modes govern, as illustrated in example problems included in Appendix A.

Per ACI (2014), anchors are sometimes embedded in narrow sections (i.e. in structural members with limited thicknesses). This case needs to be investigated whenever the edge distance c_{a2} and the thickness of the structural member are both less than $1.5c_{a1}$. In this case, the value of c_{a1} needed for computation of strength in shear loading shall not exceed the largest of:

$c_{a2}/1.5$, where c_{a2} is the largest edge distance;

$h_a/1.5$; and

$S/3$, where S is the maximum anchor spacing perpendicular to direction of shear between anchors within the anchor group with h_a being the anchor embedment length.

Tables 7.1 to 7.3 list an overview of strength design requirements for various modes of failure depicted in Figures 7.7 and 7.8 along with pertinent sections of Chapter 17 of ACI (2014) that are applicable to each mode. Note that for the modes of failure specified as concrete splitting, there is no strength design equation. The requirements for this mode are specified through limits on anchor spacing and edge distance.

Table 7.1 Anchor tension failure modes, see also, Mohammadi (2021)

Failure Mode	Illustration	Description and ACI (2014) Strength Requir
Steel element failure	Figure 7.7a	Ductile and non-ductile element Single anchor: $\phi N_{sa} \geq N_{ua}$ Individual anchor in a group: $\phi N_{sa} \geq N_{ua,i}$ Anchors as a group: not applicable
Concrete breakout	Figure 7.7b	Occurs by formation of conical concrete failure surfaces Single anchor: $\phi N_{cb} \geq N_{ua}$ Individual anchor in a group: not applicable Anchors as a group: $\phi N_{cbg} \geq N_{ua,g}$

Side-face blowout	Figure 7.7c	Likely to occur in deep embedment situations at close edge dist Single anchor: $\phi N_{sb} \geq N_{ua}$ Individual anchor in a group: not applicable Anchors as a group: $\phi N_{sbg} \geq N_{ua,g}$
Splitting	Figure 7.7d	Likely to occur in deep embedment situations at close edge dist requirement, anchors must satisfy spacing, thickness, and edge
Pullout	Figure 7.7e	Can occur in both mechanical and cast-in anchors Single anchor: $\phi N_{pn} \geq N_{ua}$ Individual anchor in a group: $\phi N_{pn} \geq N_{ua,i}$ Anchors as a group: not applicable
Bond failure (adhesive anchors)	Figure 7.7f	Can occur in both mechanical and cast-in anchors Single anchor: $\phi N_a \geq N_{ua}$ Individual anchor in a group: not applicable Anchors as a group: $\phi N_{ag} \geq N_{ua,g}$

In Table 7.1,

N_{ua} is the factored applied tensile force on a single anchor.

$N_{ua,i}$ is the factored applied tensile force on an individual anchor in a group of anchors.

$N_{ua,g}$ is the factored applied tensile force on a group of anchors.

ϕN_{sa} is the factored tensile steel strength in tension for a single anchor.

ϕN_{sb} is the factored concrete side-face blowout strength in tension for a single anchor.

$\phi N_{sb,g}$ is the factored concrete side-face blowout strength in tension for a group of anchors.

ϕN_{cb} is the factored concrete breakout strength in tension for a single anchor.

$\phi N_{cb,g}$ is the factored concrete breakout strength in tension for a group of anchors.

ϕN_{sa} is the factored tensile steel strength in tension for a single anchor.

ϕN_{pn} is the factored concrete pullout strength in tension for a single anchor.

ϕN_a is the factored bond strength of a single adhesive anchor in tension.

$\phi N_{a,g}$ is the factored bond strength a group of adhesive anchors in tension.

ϕ is the strength reduction factors per Section 17.3.3 of ACI 318, and to be used along with load combination and load

factors stipulated in Section 5.3, of ACI (2014) and as summarized in Table 7.3.

Depending on the mode of failure, as indicated in Table 7.1, the computation of the strength for tension is done based on anchor's embedment length and outside diameter, its cross-sectional areas resisting bearing, anchor's metal tensile capacity, spacing and edge distance and the type and compressive strength of concrete. The reader is referred to the ACI sections listed in Table 7.1 for detailed descriptions of equations used for strength computations for the modes of failure depicted in Figure 7.7. Also, Appendix A contains example problems where the ACI strength equations are explained for a specific application problem.

Table 7.2 Anchor shear failure modes

Failure Mode	Illustration	Description and ACI (2014) Strength Requirement
Steel element failure	Figure7.8a	Occurs in both ductile and non-ductile elements Single anchor: $\phi V_{sa} \geq V_{ua}$ Individual anchor in a group: $\phi V_{sa} \geq V_{ua,i}$ Anchors as a group: not applicable
Concrete breakout	Figure 7.8b	Occurs by formation of conical concrete failure surfaces toward the edg Single anchor: $\phi V_{cb} \geq V_{ua}$ Individual anchor in a group: not applicable Anchors as a group: $\phi V_{cbg} \geq V_{ua,g}$
Concrete pryout	Figure 7.8c	Occurs with formation of a concrete failure behind anchor Single anchor: $\phi V_{cp} \geq V_{ua}$ Individual anchor in a group: not applicable Anchors as a group: $\phi V_{cpg} \geq V_{ua,g}$

In table 7.2,

V_{ua} is the factored applied shear on a single anchor.

$V_{ua,i}$ is the factored applied shear on an individual anchor in a group.

$V_{ua,g}$ is the factored applied shear on a group of anchors.

ϕV_{sa} is the factored steel strength in shear for a single anchor.

ϕV_{cb} is the factored concrete breakout strength in shear for a single anchor.

ϕV_{cbg} is the factored concrete breakout strength in shear for a group of anchors

ϕV_{cp} is the factored concrete pryout strength in shear for a single anchor.

ϕV_{cpg} is the factored concrete pryout strength in shear for a group of anchors.

The reader is referred to the ACI sections listed in Table 7.2 for detailed descriptions of equations used for strength computations for the shear modes of failure. Also, Appendix A contains example problems where the ACI strength equations are explained for a specific application.

The strength reduction factor ϕ for shear strengths is also summarized in Table 7.3. Again, depending on the mode of failure, as indicated in Table 7.2, the computation of the strength for shear is done based on anchor's embedment length and outside diameter, its cross-sectional areas, anchor's metal shear capacity, spacing and edge distance, and the type and compressive strength of concrete. Once the strength values for both tension and shear are computed, the combined effect of tension and shear will need to be considered to check the design using some type of interaction equation (as described later in this section).

Table 7.3 Strength reduction factor ϕ per Section 17.3.3 of ACI (2014)

Situation	ϕ
Anchors governed by strength of a ductile steel element	0.75 (for tension loads) 0.65 (for shear loads)
Anchors governed by strength of a brittle steel element	0.65 (for tension loads) 0.60 (for shear loads)
Anchors governed by concrete breakout, side-face blowout, pullout or pryout strength for shear loads and tension loads (cast-in headed studs, headed bolts, or hooked bolts) for two conditions (A and B) as described below	0.75 (Condition A) 0.70 (Condition B)
Post-installed anchors (for Categories 1, 2, or 3 as described below)	0.75 (Condition A, & Category 1) 0.65 (Condition B, & Category 1) 0.65 (Condition A, & Category 2) 0.55 (Condition B, & Category 2) 0.55 (Condition A, & Category 3) 0.45 (Condition B, & Category 3)
Notes: (1) Conditions A and B refer to anchors that are governed by more brittle concrete breakout or blowout failure modes. If supplen reinforcement is present (except for pullout and pryout strengths), Condition A is used. If supplementary reinforcement is not pr for pullout or pryout strengths, Condition B is used.	
(2) Category 1: low sensitivity to installation and high reliability; Category 2: medium sensitivity to installation and medium reliab Category 3: high sensitivity to installation and lower reliability. These categories are determined by tests per ACI 355.2 (2007), mechanical anchors, and ACI 355.4 (2011), for adhesive anchors.	

Provisions for anchors subject to seismic loading – For anchors in seismic design category (SDC) C, D, E, or F, additional design requirements will need to be satisfied as outlined in Sections 17.2.3.2 through 17.2.3.7 of ACI (2014). In cases where post-installed anchors are used, the ACI-prescribed prequalification of anchors to resist earthquake loads per ACI 355.2 (2007) and 355.4 (2011) will also need to be met. For anchors in tension, seismic design requirements for anchors are primarily intended to assure the ductility demand. In essence, this requirement calls for a design in which the concrete-governed strength for a single anchor is greater than the steel strength of the anchor. For a group of anchors, the ratio of the tensile load on the most highly stressed anchor to the steel strength of that anchor shall be equal to or greater than the ratio of the tensile load on tension loaded anchors to the concrete-governed strength of the anchor group.

A summary of seismic design requirements for anchors embedded in concrete is provided in Table 7.4 with reference to pertinent sections of ACI (2014).

Table 7.4 Anchor seismic design requirements, ACI (2014)

Design Requirement	Description
Seismic Design Category	Provisions are applicable to SDC C, D, E, or F
Tensile loading	If the tensile component of the strength level earthquake force on a sing anchors is less than or equal to 20% of the total factored anchor tensile same load combination, the design can be based on Table 7.1 with consi requirement as described below.
Tensile loading	If the tensile component of the strength level earthquake force on a sing anchors is greater than 20% of the total factored anchor tensile force as load combination, anchors and their attachment shall be designed per re in Table 7.5.
Ductility	One of four options, designated as (a) through (d) shall be satisfied as d
Shear loading	If the shear component of the strength level earthquake force applied to anchors is less than or equal to 20% of the total factored anchor shear f same load combination, the design will need to satisfy the shear strengt 7.2.
Shear loading	If the shear component of the strength level earthquake force applied to anchors is greater than 20% of the total factored anchor shear force ass load combination, the design will need to satisfy the requirement of Tabl

Seismic ductility requirement – The main trust of ductility requirement is that the design is governed by yielding of the ductile steel element of the anchor. As indicated in ACI (2014), if the anchor cannot meet the specified ductility requirements (see Table 7.6), then the attachment should be either designed to yield (in the case of structural or light gauge steel), or designed to crush (in the case the attachment is a wood member). In cases where the ductility requirement is satisfied, then any attachments to the anchor should be designed not to yield. In designing attachments using yield mechanisms to provide the ductility needed, the ratio of specified yield strength to the expected strength for the material of the attachment should be considered in determining the design force. The value used for the expected strength should consider both material over-strength and strain hardening effects. ACI (2014) indicates that for attachment made up of steel, if the “specified yield strength” is the only material information

available, the expected strength is recommended to be taken as approximately 1.5 times the specified yield strength. However, if the actual yield strength of the steel is known, the expected strength is recommended to be taken as approximately 1.25 times the actual yield strength. It is also noted that under seismic conditions, and as a safe design condition, the full shear force should be assumed in any direction.

*Table 7.5 Anchor tensile design strength for seismic resistance, ACI (2014)**

Mode of Failure	Description and ACI (2014) Strength Requirement	
(a) Steel strength in tension	ϕN_{sa} is used for a single anchor, or for the most highly stressed individual anchor in a group of anchors.	17.
(b) Concrete breakout strength in tension	$0.75\phi N_{cb}$ or $0.75\phi N_{cbg}$ are used (if anchor reinforcement at both sides of the breakout surface is developed, N_{cb} or N_{cbg} need not be computed, and the design strength of the anchor reinforcement is used instead of the concrete breakout strength).	17.
(c) Pullout strength in tension	$0.75\phi N_{pn}$ is used for a single anchor, or for the most highly stressed individual anchor in a group of anchors.	17.
(d) Concrete side-face blowout strength in tension	$0.75\phi N_{sb}$ or $0.75\phi N_{sbg}$ are used.	17.
(e) Bond strength of adhesive anchor in tension	$0.75\phi N_{a}$ or $0.75\phi N_{ag}$ are used.	17.
* For all strength computations, the concrete is considered cracked unless it can be demonstrated otherwise.		

Table 7.6 Anchor ductility requirements for seismic design ACI (2014)

Option	Requirement

(a)	For single anchors, concrete-governed strength needs to be greater than the steel strength of the anch For anchor groups, ratio of the tensile load on the most highly stressed anchor to the steel strength of needs to be equal to or greater than the ratio of the tensile load on tension loaded anchors to the conc governed strength of those anchors. In each case: (i) The steel strength is taken as 1.2 times the nominal steel strength of the anchor. (ii) The concrete-governed strength taken to be equal to the nominal strength considering pullout, side blowout, concrete breakout, and bond strength as applicable. For pullout in groups, the ratio is calculat most highly stressed anchor. (iii) Anchors shall transmit tensile loads via a ductile steel element with a stretch length $8d_b$ (unless oth determined by analysis). (iv) Where anchors are subject to load reversals, they need to be protected against buckling. (v) Where connections are threaded and the ductile steel elements are not threaded over their entire le ratio of ultimate to yield strength needs to be greater than 1.3 unless the threaded portions are upset (portion not included in the stretch length). (vi) Deformed reinforcing bars used as ductile steel elements to resist earthquake effects are limited to Grades 40 and 60 or ASTM A706 Grade 60.
(b)	The anchor or anchor group to be designed for the maximum tension that can be transmitted to them l development of a ductile yield mechanism in the attachment in tension, flexure, shear, or bearing, or a of these conditions, and considering both material over-strength and strain hardening effects for the at (anchor design tensile strength per Table 7.5).
(c)	The anchor or group of anchors to be designed for the maximum tension that can be transmitted to the a non-yielding attachment (anchor design tensile strength per Table 7.5).
(d)	The anchor or group of anchors to be designed for the maximum tension obtained from design load co that include E, with E amplified by the Ω_0 factor* (anchor design tensile strength per Table 7.5).
* E indicates the effect of earthquake-induced forces; and Ω_0 is the amplification factor to account for over-strength of the seis general building code.	

Table 7.7 Anchor shear design requirements for seismic effect ACI (2014)

Condition	Requirement

(a)	Anchors and their attachments to be designed using one of options (i) through (iii): (i) The anchor or group of anchors shall be designed for the maximum shear that can be transmitte group of anchors based on the development of a ductile yield mechanism in the attachment in flexu or a combination of those conditions, and considering both material over-strength and strain harde attachment. (ii) The anchor or group of anchors shall be designed for the maximum shear that can be transmitte non-yielding attachment. (iii) The anchor or group of anchors shall be designed for the maximum shear obtained from design that include E, with E amplified by the Ω_0 factor* (anchor design shear strength per table 7.2).
(b)	Where anchor reinforcement is provided in accordance with 17.5.2.9, no reduction in design shear s specified in 17.5.2.9 shall be required.
(c)	Single anchors or groups of anchors that are subjected to both tension and shear forces shall be de requirements of 17.6, with the anchor design tensile strength calculated from 17.2.3.4.4.
(d)	Anchor reinforcement used in structures assigned to SDC C, D, E, or F shall be deformed reinforcem limited to ASTM A615 Grades 40 and 60 satisfying the requirements of 20.2.2.5(b) (i) and (ii) or AS
* E indicates the effect of earthquake-induced forces; and Ω_0 = amplification factor to account for over-strength of the seismic-general building code	

There are also additional requirements for seismic design outlined in ACI 2014 including provisions for adhesive anchors and cases where lightweight concrete is used. The reduction factors for strength computations are recommended for lightweight concrete as 0.8 for expansion and adhesive anchor concrete failure mode and 0.6 for adhesive anchor bond failure in tension. For cast-in and undercut anchor concrete failure modes, no reduction is considered.

Interaction of tension and shear forces – In most applications, anchors or groups of anchors are subject to both tension and shear forces. ACI (2014) requires that the interaction of the tension and shear forces is considered in design of anchors in cases when

$$\frac{V_{ua}}{\phi V_n} > 0.2 \tag{7.3.1.1}$$

and

$$\frac{N_{ua}}{\phi N_n} > 0.2 \tag{7.3.1.2}$$

In which the values of ϕV_n and ϕN_n are obtained using the ACI (2014) procedures following the modes of failures and strength requirements summarized in Tables 7.1 to 7.7. It is noted that if the ratio $V_{ua}/(\phi V_n)$ is less than or equal to 0.2, the full strength in tension can be used (i.e., $\phi N_n \geq N_{ua}$) in the interaction equation. Similarly, if the ratio $N_{ua}/(\phi N_n)$ is less than or equal to 0.2, the full strength in shear can be used (i.e., $\phi V_n \geq V_{ua}$) in the interaction equation. The interaction is considered using one of the following two equations:

$$(\frac{N_{ua}}{\phi N_n}) + (\frac{V_{ua}}{\phi V_n}) \leq 1.2 \tag{7.3.1.3}$$

$$(\frac{N_{ua}}{\phi N_n})^{5/3} + (\frac{V_{ua}}{\phi V_n})^{5/3} \leq 1.0 \tag{7.3.1.4}$$

Note, $N_{ua}/(\phi N_n)$ and $V_{ua}/(\phi V_n)$ are referred to as the "strength utilization ratios."

Illustrative Example – See Example 7.3 in Appendix A for an illustration of the connection design of a shelf angle to concrete along with pertinent ACI anchor strength equations.

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7.3.2: Shelf angle connection to masonry using anchors

Design of shelf-angle connections to masonry walls follows the recommendations of ACI 530 (2013). Additional guidance specific to brick masonry and concrete masonry wall installations is provided in BIA (1986) and NCMA (2013), respectively. The strength of anchors is evaluated for both tension and shear, and the adequacy of an anchor or anchor group must be verified considering the interaction between these two forces. Both the LRFD and ASD methods are currently in use. The calculation of strength values is essentially the same for both methods and is based on the potential failure mechanisms of anchors in tension and shear. Final design verification under ASD uses factors of safety, while LRFD applies load and resistance reduction factors. Special provisions for anchor design must also be considered in seismic regions.

Current design criteria require anchors to be embedded in properly reinforced, grouted masonry. Anchor bolts with a diameter of one-quarter of an inch (6.4 millimeters) may also be installed within mortar bed joints, provided the joint thickness is at least one-half of an inch (12.7 millimeters). For the purposes of applying the provisions of ACI 530 (2013), anchor bolts with a one-quarter-inch (6.4-millimeter) diameter placed in mortar joints are permitted to be treated as though they are embedded in grout. Requirements for the extent of grout surrounding the anchor, as well as the minimum clear distances between the anchor end and the masonry face, are specified in ACI 530 (2013). Additional guidance is available in BIA (1986) for brick masonry and in NCMA (2013) for concrete masonry construction.

Several useful references pertinent to this section include WSCPA (2004), WSCPA (2011) and BIA (2005).

Type of Anchors – Two broad categories of anchors are used in masonry application. These are

- Conventional anchor bolts.
- Proprietary anchor bots.

Conventional anchors bolts are made by steel fabricators to meet a specific project. They are also available in standard sizes (of length and diameter) through steel suppliers. Proprietary anchors are available through manufacturers under their brand names. These manufacturers also provide documents and often technical assistance for design.

Table 7.8 summarizes different types of anchors within these two broad categories. The types listed for proprietary anchors are generic and based on the underlying theory in how anchors work. Thus, anchor configurations, sizes, and lengths are specific to each manufacturer. This type of data can be obtained through each manufacturer's published materials.

Table 7.8 Categories and types of anchors used in masonry (BIA, 1986)

Category	Types	Configuration	
Conventional	Headed Bolts	Square or hex	ASTM A307 or A
	Bent Bar Anchors	Generally made in "J" or "L" shapes, although other types such as "U" shapes are also used.	ASTM A36
	Through Bolts	Extends through the thickness of the wall; a bearing plate is used on the surface opposite of where the shelf angle is located.	ASTM A307 for bearing plate
	Plate Anchors	A plate is welded at one end of a steel bar; while the other end is threaded for attachment.	ASTM A36 for b electrode and si on the plate thic

Proprietary	Expansion Anchors	Available as (1) sleeve; and (2) wedge anchors depending on manufacturer	Depends on ma
	Adhesive Anchors	Available as (1) anchors with pre-mix, self-contained system; and (2) the types that requires measurement and mixing of epoxy materials at the time of installation	Depends on ma

Effective anchor cross sectional area and effective embedment length – In conventional anchors, the effective cross-sectional area will need to consider the reduction because of anchor threading. Table 7.9 summarizes effective cross-sectional areas as recommended in NCMA (2013).

Table 7.9 Conventional anchor nominal and effective cross-section areas

Anchor Diameter, inches (millimeters)	Nominal Area, square inches (square millimeters)	Effective Area, square inches (squar
1/2 (13)	0.196 (126)	0.142 (92)
5/8 (16)	0.307 (198)	0.226 (146)
3/4 (19)	0.442 (285)	0.334 (215)
7/8 (22)	0.601 (388)	0.462 (298)

The effective embedment length for a conventional anchor is usually taken as four times the diameter or 2 inches (51 millimeters), whichever is greater. It is noted that the embedment length l_b for headed anchors is measured parallel to the anchor bolt axis from the surface of the masonry to the bolt head bearing surface (NCMA, 2013) as shown in Figure 7.9a. For proprietary anchors, the information on the effective cross-sectional area and embedment length is provided in each manufacturer's product technical data.

Anchor modes of failure and strength computation – Modes of failure considered in design of anchors in masonry are tension and shear; and the computation of the strength is performed with consideration for these modes. ACI 530 (2013) provides information on these modes of failure and equations for strength computations. As shown in Figures 7.9a and b, failure in tension is defined via a conical surface separated from the masonry. This surface is defined with a 45-degree angle and dimension l_b (the embedment length). For shear, mainly the failure occurs due to breakout. Another possible mode of failure is shear pryout (Figure 7.9c). Considering this mode of failure, the critical parameter is the anchor distance from the edge l_{be}, as shown in Figure 7.9c. For a single anchor, the strength computation is based on the projected area for tension, A_{pt} and for shear A_{pv}, each computed based on their corresponding conical critical surface. Referring to Figures 7.9a and b, and knowing that the angle of the conical surface is equal to 45 degrees,

$$A_{pt} = \pi {l_b}^2 \tag{7.3.2.1}$$

And from Figure 7.9c,

$$A_{pv} = \frac{\pi l_{be}^2}{2} \tag{7.3.2.2}$$

As indicated in ACI 530 (2013), both for tension and shear and in computing the respective projected areas, the portion of the projected area that overlaps an open cell, or open head joint, or that lies outside the masonry will need to be deducted from the value of A_{pt} or A_{pv}. In cases where the projected areas of adjacent anchor bolts overlap, the value of A_{pt} or A_{pv} will need to be adjusted so that no portion of masonry is included more than once. For example, for two anchors with an overlapping projected area equal to A_b, the projected area of each anchor is

$$A_{pt} = \pi l_b^2 - 0.5A_b \tag{7.3.2.3}$$

The deduction of one half of $0.5A_b$ is done so that no portion of the masonry is included more than once. This is to avoid overstresses on the masonry. In using Equation 7.3.2.1, ACI 530 (2013) defines the effective embedment length l_b as follows:

- For headed anchor bolts, the effective embedment is the length of the anchor bolt measured perpendicular from the masonry surface to the compression bearing surface of the anchor head.
- For bent-bar anchor bolts, the effective embedment is the length of the bar measured perpendicular from the masonry surface to the compression bearing surface of the bent end, minus one anchor bolt diameter. Thus, in the case of bent-bar anchor bolts, practically l_b is the length of the bar measured from the surface of the masonry to the point at the end where the curve starts.

These projected areas are important parameters in computing the tensile and shear capacity of anchor bolts. The ACI 530 (2013) also permits using tests in the determination of anchor bolt strengths. In that case, tests must be according to the ASTM E488 and a minimum of five tests will need to be conducted. The allowable anchor load values for design cannot exceed 20% of the average test values.

The following procedure is used when anchor bolt strength values are determined by calculations.

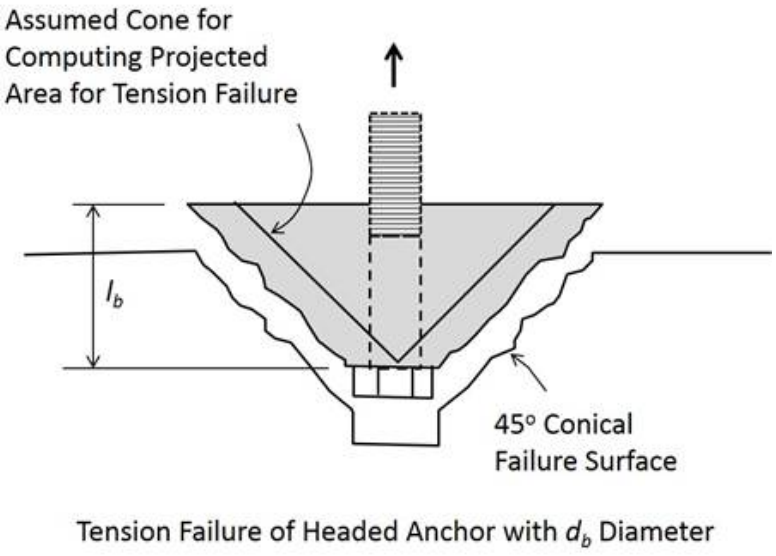


Figure 7.9a Tension failure of headed anchor

(This figure shows a cross section of failure when a conical portion of masonry is separated from the rest of the material and is attached to a headed anchor being pulled out.)

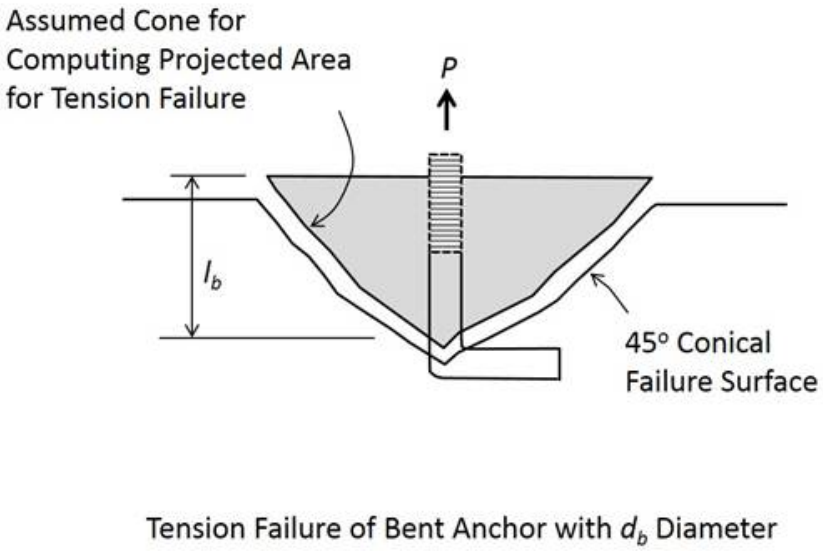


Figure 7.9b Tension failure of bent anchor

(This figure shows a cross section of failure when a conical portion of masonry is separated from the rest of the material and is attached to a bent anchor being pulled out.)

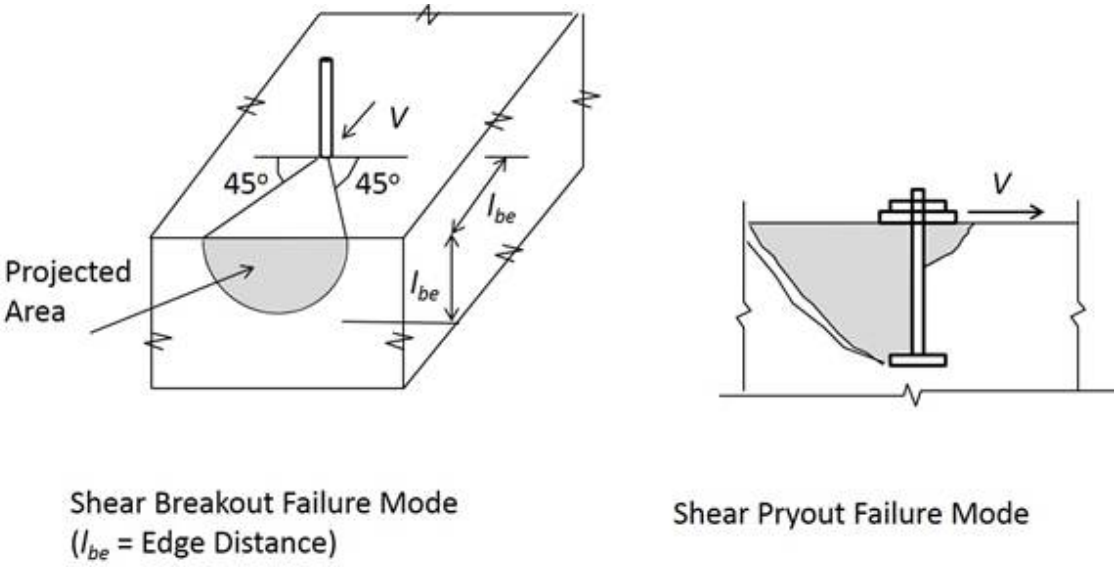


Figure 7.9c Shear breakout and pryout failure modes of anchor

(This figure shows two details. On the left an isometric image of a half conical portion of material is shown being separated because of a shear force on an anchor. On the right, a cross-section of an anchor in the material is shown with a portion of the material, with a triangular shape, is separated from the rest of the material behind the anchor.)

ASD method – In general, three resistance values, corresponding to different modes of failure, are involved in computing the allowable tensile load of a headed anchor B_a. These are

B_{ab}: the concrete breakout resistance

B_{ap}: the pull-out resistance

B_{as}: the steel yielding resistance

In headed anchors, only the concrete breakout and the seel yielding are the prevalent modes of failure; and as such B_a is taken as the smaller of B_{ab} and B_{as}. However, for bent anchors, all three modes are involved; and as such, B_a is taken as the smallest of the three resistances B_{ab}, B_{ap}, and B_{as}. The equations specified in the ACI 530 for computing theses three resistances are provided below.

$$B_{ab} = 1.25 A_{pt} \sqrt{f'_m} \tag{7.3.2.4}$$

$$B_{ap} = 0.6 f'_m e_b d_b + 120\pi (l_b + e_b + d_b) d_b \tag{7.3.2.5}$$

$$B_{as} = 0.6 A_b f_y \tag{7.3.2.6}$$

Equation 7.3.2.5 is based on the anchor pull-out strength and is used for bent anchors only. In this equation d_b is the bar diameter and e_b is the projected leg extension of the bent bar anchor, measured from inside the edge of the anchor at the bend to farthest point of the anchor in the plane of the hook.

The allowable shear load of an anchor embedded in grout for either headed or bent-bar anchors B_v is the smallest of the following four values.

$$B_{vb} = 1.25 A_{pv} \sqrt{f'_m} \tag{7.3.2.7}$$

$$B_{vc} = 350 \sqrt[4]{f'_m A_b} \tag{7.3.2.8}$$

$$B_{vpry} = 2.0 B_{ab} = 2.5 A_{pt} \sqrt{f'_m} \tag{7.3.2.9}$$

$$B_{vs} = 0.36 A_b f_y \tag{7.3.2.10}$$

Equation 7.3.2.7 defines the allowable shear strength governed by masonry breakout; Equation 7.3.2.8 is based on the masonry crushing; Equation 7.3.2.9 uses the anchor pryout; and Equation 7.3.2.10 defines the allowable shear strength governed by steel yielding.

The combined effect of the axial load and shear will need to satisfy the following inequality.

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \leq 1 \tag{7.3.2.11}$$

In which b_a and b_v are applied tension and shear on the anchor, respectively.

LRFD method – When using the LRFD design method, applicable capacity reduction factor (ϕ) and load factors are considered. The nominal axial tensile strength B_{an} of a headed anchor bolt is the smaller of the following two values. The factored (i.e., design) axial tensile strength is ϕB_{an}.

$$B_{anb} = 4.0 A_{pt} \sqrt{f'_m} \tag{7.3.2.12}$$

$$B_{ans} = A_b f_y \tag{7.3.2.13}$$

Equation 7.3.2.12 is based on the masonry breakout strength; while Equation 7.3.2.13 uses the anchor bolt strength in yielding.

The nominal tensile strength of an anchor with bent B_{an} is the smallest of the following three values. The design tensile strength is taken as ϕB_{an}.

$$B_{anb} = 4.0 A_{pt} \sqrt{f'_m} \tag{7.3.2.14}$$

$$B_{anp} = 1.5 f'_m e_b d_b + 300\pi (l_b + e_b + d_b) d_b \tag{7.3.2.15}$$

$$B_{ans} = A_b f_y \tag{7.3.2.16}$$

Equation 7.3.215 is based on the anchor pull-out strength.

The nominal shear strength of an anchor embedded in grout, for either headed or bent anchors B_{nv}, is the smallest of the following four values. The factored shear strength value is ϕB_{nv}.

$$B_{vnb} = 4.0 A_{pv} \sqrt{f'_m} \tag{7.3.2.17}$$

$$B_{vnc} = 1,050 \sqrt[4]{f'_m A_b} \tag{7.3.2.18}$$

$$B_{vnpry} = 2.0 B_{anb} = 8.0 A_{pt} \sqrt{f'_m} \tag{7.3.2.19}$$

$$B_{vns} = 0.6 A_b f_y \tag{7.3.2.20}$$

Equation 7.3.2.17 defines the shear strength governed by masonry breakout; Equation 7.3.2.18 is based on the masonry crushing; Equation 7.3.2.19 uses the anchor pryout; and Equation 7.3.2.20 defines the shear strength governed by steel yielding.

The combined effect of the axial tension load and shear will need to satisfy the following inequality.

$$\frac{b_{af}}{\phi B_{an}} + \frac{b_{vf}}{\phi B_{nv}} \leq 1 \tag{7.3.2.21}$$

In which b_{af} and b_{vf} are factored applied tension and shear on the anchor, respectively. The capacity reduction factor ϕ is taken as 0.9 for tension and 0.8 for shear.

In situations where an anchor nominal strength is determined by tests, the strength used for design is limited to 65 percent of the average strength values from the tests.

Illustrative example – See Example 7.4 in Appendix A for an illustration of anchor design connecting a shelf angle to a masonry wall.

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7.3.3: Shelf angle connection to structural steel

In this situation, steel columns provide support for the shelf angle. Figure 7.10 presents a suggested detail using a flat plate or an L-shaped bracket as support for the shelf angle. The plate and bracket are welded to the column, while the shelf angle is connected to the supporting plate or bracket with bolts for easy installation. Notice that there is a beam that runs in parallel with the shelf angle spanning between two adjacent columns. This beam is intended to support the back or infill wall behind the veneer. The cavity between this wall and veneer is still kept as usual with two-piece anchors attaching the veneer to the back wall as discussed earlier.

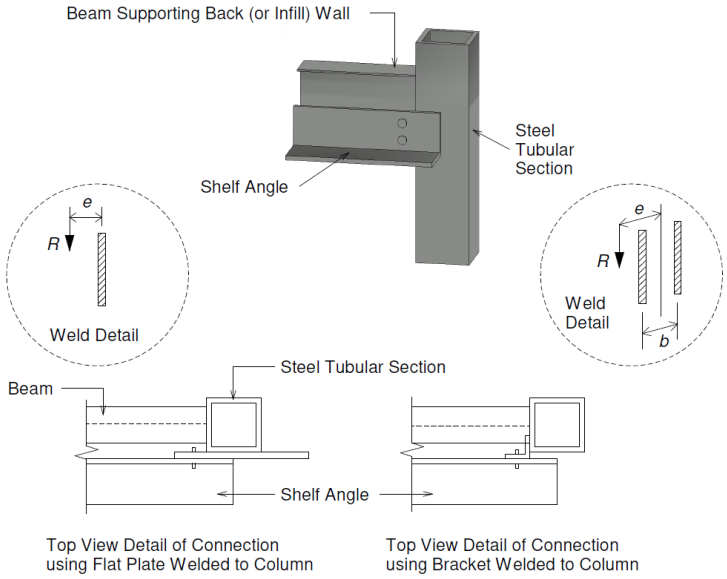


Figure 7.10 An example of shelf angle supported by steel columns

(This figure shows an isometric detail of a shelf angle attached to a tubular steel column. Below this image, on the left, a cross-section of the detail, viewed from the top, is shown with a flat plate welded to the column and bolted to the shelf angle. On the right, the same cross-section is shown, however, the shelf angle connection to the column is via an angle-bracket that is welded to the column and bolted to the shelf angle.)

Considering structural design, it is noted that the angle must now act as a simply-supported beam spanning over a distance L (which is nearly equal to the spacing between two adjacent columns). Thus, depending on the size of the angle, the span length will be limited. And as such, in cases where adjacent main load-bearing columns are relatively far from each other, we may have to install additional columns (sometimes referred to as soldier columns) in the perimeter of the structure to maintain the limitation on the span length L. The load on the angle can conservatively be taken as the load of the masonry with a height equal to h and neglecting any interlock action effect of the masonry blocks supported by the angle. In this way, the bending moment and shear are $M_u = q_u L^2/8$ and $V_u = q_u L/2$, respectively, where q_u is the factored applied load when using the LRFD method. Thus, an appropriate angle size will need to be selected to satisfy the bending moment and shear design. However, in most cases, the bending moment is the determining factor in design. And as such, the design for bending is satisfactory when $M_u \leq \phi M_n$. The nominal bending capacity M_n of the angle is computed using the AISC procedure with consideration for

- The yielding capacity of the angle and the fact that the toe in the vertical leg is under compression;
- The unsupported length of the angle (which is practically the span length L in most situations) and procedure for lateral torsional buckling; and
- The local buckling effect which is controlled with the ratio b/t (in which b is the width of the angle and t is its thickness).

The procedure in computing M_n is explained later in this section.

Another consideration in the selection of the size of the angle is the number of bolts needed for the connection to the plate or bracket. With two bolts considered in the detail in Figure 7.10, the minimum size of the angle's vertical length will be 6 inches

(152 millimeters).

The design of the bolts and welds, connecting the support plate or bracket to the column, follows AISC (2017). Bolts are primarily under a shear force equal to the factored reaction R_u of the shelf angle at each end, when LRFD is used. Thus, with two bolts used, the total design shear resistance $\phi R_n = 2\phi_v r_n$, which will need to be larger than R_u for a satisfactory design. For example, for A325 bolts with 5-8th of an inch (16 millimeters) of diameter, and with the thread included in the shear plane, $\phi_v r_n$ = 11.0 kips (48.9 kilonewtons).

In the example configuration of Figure 7.10, the weld details for the two possible connection types are also shown. As shown, a key parameter for design is the eccentricity e, which is measured from the center of the bolts to the center of the weld or the weld group. Tables 8-4 of AISC provide a convenient means of computing the weld resistance. Per AISC, the weld resistance is

$$\phi R_n = 0.75 \times C C_1 D l \tag{7.3.3.1}$$

in which,

l is the weld length.

C is a factor from Table 8-4 of AISC based on the ratio e/l and a factor k – where k/l represents the distance between two vertical weld strips (note that k is equal to zero, if only one weld strip is used).

C_1 is a factor depending on the type of electrode used in making the weld and is equal to 1 for E70 electrode.

D is the number of sixteenth-of-an-inch fillet wed size (for example, if the weld size is 3-16th of an inch in size, then this factor is equal to 3).

The weld design is satisfactory when $R_u \leq \phi R_n$.

In addition to checking the design of the shelf angle for bending and shear, it is important to check the deflection y for the service load q, which is usually the dead weight of the masonry supported by the angle using the following equation for a simply-supported flexural element.

$$y = \frac{5qL^4}{384EI} \tag{7.3.3.2}$$

in which,

E is the modulus of elasticity.

I is the moment of inertia of the angle cross-section.

This deflection is limited to $L/600$ and 3-8th of an inch (9.5 millimeters), whichever is smaller.

Step-by-step procedure for computing M_n for a shelf angle spanning over a length L – We start by selecting the angle size, for which the section modulus (S_x), the width (b), the thickness (t), ultimate strength (F_u) and yield strength (F_y) are known. For most angles, F_y = 36, and F_u = 58 kips per square inch (or 248.2 and 399.9 megapascals, respectively).

Then we compute three possible values for M_n and select the smallest value. Please note that this procedure ignores any torsion that is produced because of the fact that the applied load does not exactly act at the shear center of the angle cross section. Furthermore, the angle is considered restrained at ends only, thus the unsupported length is L. This procedure is based on the 15th edition of AISC (2017). Use kips and inches in all equations.

Step 1. Compute M_n based on flexural yielding

$$M_n = 1.5 M_y \tag{7.3.3.3}$$

Where in this case, $M_y = S_x F_y$, thus

$$M_n = 1.5 S_x F_y \tag{7.3.3.4}$$

Step 2. Compute M_n based on the lateral torsional buckling

For these computations, take $M_y = 0.80 S_x F_y$, and first compute the critical moment M_{cr} from the following equation.

$$M_{cr} = \frac{0.58 E b^4 t C_b}{L_b^2} [\sqrt{1 + 0.88 (\frac{L_b t}{b^2})^2} - 1] \tag{7.3.3.5}$$

In this equation, take C_b = 1.14, and by doing so, we are considering lateral restraints at ends only.

If the ratio $\frac{M_y}{M_{cr}}$ is less than or equal to 1, then

$$M_n = [1.92 - 1.17 \sqrt{\frac{M_y}{M_{cr}}}] M_y \leq 1.5 M_y \tag{7.3.3.6}$$

Note: in this equation $M_y = 0.80 S_x F_y$

If the ratio $\frac{M_y}{M_{cr}}$ is greater than 1, then

$$M_n = [0.92 - \frac{0.17}{(M_y / M_{cr})}] M_{cr} \tag{7.3.3.7}$$

Step 3. Compute M_n based on the local buckling effect

First compute the parameter $\lambda = b/t$, then compute the limiting non-compact slenderness ratios

$\lambda_p = 0.54 \sqrt{E/F_y}$

and

$\lambda_r = 0.91 \sqrt{E/F_y}$

If λ is less than λ_p, the section is compact, local buckling does not apply, select your final M_n as the minimum of those in Steps 1 and 2.

If λ is larger than λ_p but less than or equal to λ_r, the leg is non-compact, then

$$M_n = 0.8 F_y S_x [2.43 - 1.72 (b/t) \sqrt{F_y / E}] \tag{7.3.3.8}$$

If If λ is greater than λ_r, the section has a slender leg, then

$$M_n = 0.8 S_x [\frac{0.71 E}{\lambda^2}] \tag{7.3.3.9}$$

The final M_n is the smallest values from Steps 1, 2 and 3.

Illustrative Example – Example 7.5 in Appendix A illustrates the application of equations and materials presented in this section.

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7.4: Lintel Design

Lintels are structural elements used above openings in walls to support the weight of the masonry. If the span length is not very long, a structural member made up from a single steel angle profile may be used as a lintel. For longer span lengths, channels, W-sections, and even built-up sections made up of plates along with other profiles have also been employed as lintels. In certain applications, reinforced concrete members with rectangular cross-sections, timber, and masonry have also been used as lintels. Figure 7.11 shows a lintel supported by brick masonry with flashing. The lintel carries the load of the masonry built above it.

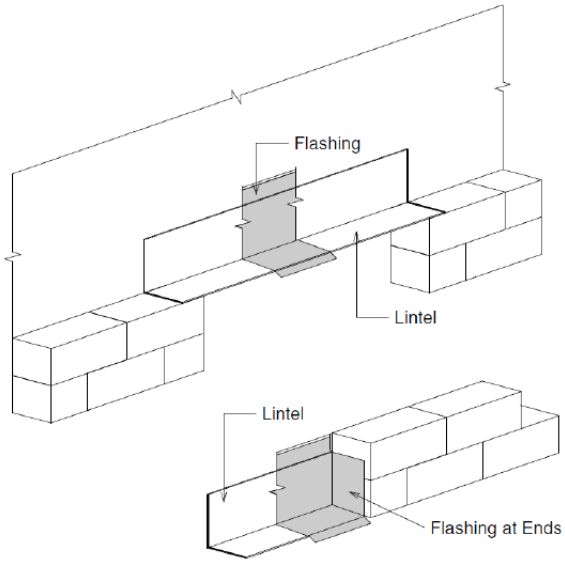


Figure 7.11 Detail of a lintel with flashing

(This figure shows an isometric view of a portion of a masonry wall with am opening, which is spanned by an angle Labeled as lintel. A portion of flashing that goes over the angle is also shown.)

Design requirements – Lintel design requires attention to both architectural and structural considerations. From an architectural standpoint, critical factors include the proper detailing of flashing, weep holes, and wicks to ensure effective drainage of water and moisture. The flashing (see Figure 7.11) must extend the full length of the lintel and be turned up at both ends to form dam-like barriers, thereby preventing water infiltration from the sides. From a structural perspective, design involves identifying the applied loads and performing the necessary structural analysis to verify the adequacy of the lintel. The lintel will be considered as a simply-supported beam. The loads applied on a lintel include

- The weight of the masonry supported by the lintel.
- Any load (dead load and live load) from the floor, immediately above the lintel, or roof.
- Self-weight of the the lintel.

Loads – Depending on the height of masonry above the lintel, the load transferred to the lintel, from the weight of masonry, may be either uniform or triangular. Figure 7.12 provides a summary of all applicable loads on a lintel. The description of these loads is provided below.

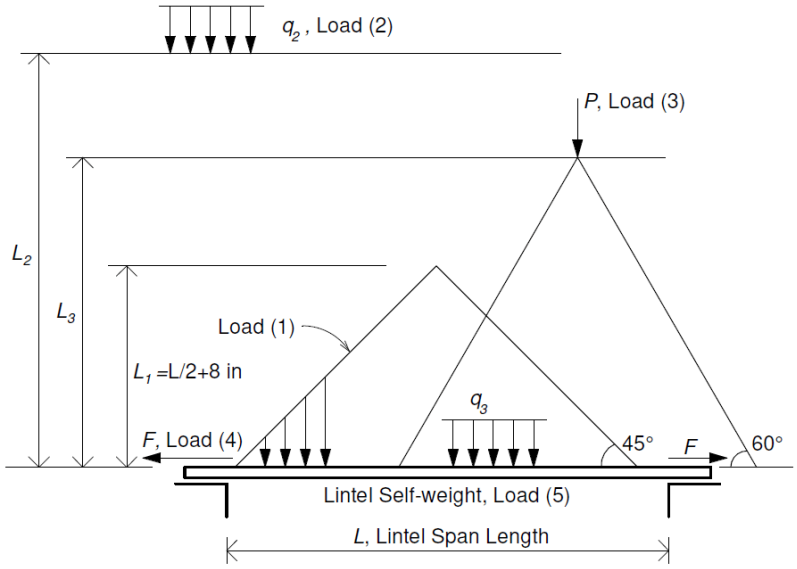


Figure 7.12 Typical load applied on a lintel

(This figure shows a simply-supported beam with five different loads on it. Load 1 is shown as a 45-degree isosceles triangle with a height of L_1 depicting the distributed load from the weight of the masonry above. Load 2 is a distributed load at L_2 distance above the beam. Load 3 is shown as an equilateral triangle depicting a distributed load from a concentrated load applied at L_3 distance above the beam. Load 4 is a pair of two horizontal forces acting in opposite directions of one another at the ends of the beam. Load 5 is a distributed load depicting the self-weight of the beam.)

Load (1): This is the main loading on the lintel. The triangular distribution forms when the height of the masonry above the lintel is at least equal to $L/2$ plus 8 inches (203 millimeters), where L is the span length. Furthermore, the triangular distribution can only be used when the masonry at the top of the lintel forms an in-plane arching action. This happens when there is adequate interlock resistance between masonry blocks and mortars. In situations where prefabricated masonry blocks are used to construct the façade above the lintel, the structural integrity required to form the arching action may not be present. In this situation, and also in cases where the masonry above the lintel has a height less than $L/2$ plus 8 inches (203 millimeters), it is more appropriate to simply apply the entire weight of the masonry above the lintel in the form of a uniformly distributed load as a conservative assumption.

Load (2): This refers to the dead and live loads from the floor or roof above the lintel. If the line of action of these loads is located at a height greater than $L/2$ plus 8 inches (203 millimeters), they may be neglected; otherwise, the full load is assumed to be transferred to the lintel.

Load (3): Any concentrated load applied at a height of L_3 will transfer to the lintel as a distributed load with a 60-degree angle (Figure 7.12). A concentrated load may be because of a post supported by the wall above the lintel, a flagpole, a frame for a commercial sign, and so on. Using L_3 as the height at which the load P applies, and with the width of the wall as T_E, the distributed force from the load P on the lintel can be obtained by computing the side length of the equal triangle formed. The load over the unit area is equal to $0.87P/(T_E L_3)$. Thus, the uniform load on the lintel (using pounds per foot or kilonewtons per meter as the unit) is $q_3 = 0.87P/L_3$. Notice that depending on the location of P, only a portion of the lintel may be subjected to this distributed load.

Load (4): This is the thrust force F shown in Figure 7.12 and is developed because of the arching action developed as a result of masonry forming an arch above the lintel. In most cases it can be neglected; and usually, it may be critical if the load above the lintel is very heavy. To determine F in these situations, a detailed structural analysis is required considering an arch above the lintel creating the thrust force.

Load (5): This is the self-weight of the lintel and is considered as a uniform load.

When in-plane arching action is considered in structural analysis, temporary shoring is required to support the lintel during construction until the masonry attains sufficient strength to carry its self-weight. The minimum recommended shoring duration is 24 hours. If the lintel is subjected to superimposed loads, however, shoring should remain in place for a minimum of three days.

Additional curing time is necessary during cold-weather construction, which may further extend the required shoring period. It should also be noted that when a uniform load distribution is assumed (rather than a triangular distribution, see Load (1)), shoring is not required (BIA, 1987).

Structural analysis – The structural analysis for the lintel includes computing maximum bending moment and shear for factored load, computing the adequacy of support area to accommodate the resistance capacity of masonry and computing the deflection. The deflection is limited to $L/600$ or 0.3 inches (8 millimeters), whichever is smaller (BIA, 1987), to avoid crack development in the masonry above the lintel. In computing the deflection, the combined superimposed live and dead load must be considered. In situations where torsion is present, the lintel must also be checked to make sure the torsional rotation is limited to one 16th of an inch (1.6 millimeters). This rotation over one half of the span length of the lintel will provide the limit on the angle of twist.

Structural dynamic analysis – In certain cases, a lintel may be subjected to dynamic loading. This condition typically arises when equipment installed on the roof or on the floor above the lintel generates vibration. Such vibration induces fluctuating deflections in the lintel, which in turn produce cyclic stresses in both the lintel and the supported masonry. While these stress variations may not be critical to the lintel itself, the resulting tension–compression cycles within the masonry can lead to crack initiation and propagation. To mitigate this risk, the lintel must be designed to avoid resonance with the vibration source, thereby limiting stress cycling in the masonry. This generally requires calculation of the fundamental frequency of the lintel and ensuring that it does not coincide with the excitation frequency of the imposed vibration. Additional discussion on this topic is provided in Section 8.6.1 of Chapter 8.

Illustrative Examples – Examples 7.7 and 7.7 in Appendix A demonstrate the application of equations and materials presented in this section for lintel design. Example 8.3 in Appendix A presents an illustration of computing the natural frequency of vibration for a beam.

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7.5: Shelf Angle and Lintel Damage Issues

A common issue with both shelf angles and lintels is corrosion, which is resulted from water infiltration and moisture that have not been properly managed and controlled. As described in Chapter 2, this stems from improper, or lack of, flashing and weep holes. Other issues pertaining to shelf angles and lintels include:

- Improper design, and structural detailing of shelf angles and lintels. These problems cause excessive deflections and result in cracks in masonry walls supported by these structural elements (see, for example, Figure 2.9b).
- Inadequate, or deteriorated of, the moisture control system including flashing, weep holes, and wicks. This will cause corrosion of the shelf angle or lintel (as shown in Figure 7.1).
- Inadequate fasteners securing the shelf angle to the structural system. This can be the result of corroded anchors, anchors that have failed due to loss of bond resistance with concrete or masonry, or inadequate design of size and anchor spacing resulting in wall undergoing excessive tensile stress in mortar joints and cracks in the wall.
- Lack of adequate gap between the shelf angle and the masonry below. As shown in Figure 7.2, a small gap below the shelf angle is required, which will need to be detailed properly with backer rod and sealant. In certain cases, shelf angles lack this gap or the gap is filled with mortar, whether in the original detailing or after repair, causing a hard joint below the angle. This nearly always results in crack and further damage because of moisture infiltrating into the affected area.

In many cases, these distress conditions can only be detected by removing the wall segments around the area, which allows access to the shelf angle or lintel for inspection. If the corrosion has not significantly reduced the cross-sectional area, and calculations confirm that the section can still bear the load and minimize deflection, the repair involves cleaning the rust down to the bare metal, applying a corrosion inhibitor, and replacing connecting anchors (in the case of a shelf angle) as necessary. Proper flashing, sealant, and backer rod must be installed to prevent future damage. However, if the corrosion is extensive or the structural member's cross-section is too weakened, the repair will require replacing the old system with a new structural member. This also includes installing adequate connections, proper flashing details, backer rods, sealant (as shown in Figure 7.2), and ensuring the ends of the flashing are designed to create a moisture barrier (as shown in Figure 7.11) to prevent water infiltration at the ends of the shelf angle and lintel. Figure 7.13 shows a photo of flashing being installed for a shelf angle.



Figure 7.13 Flashing being installed on a shelf angle (Photo courtesy of Dearborn Engineering Corporation)

(This figure shows a photo of a membrane installed on a shelf angle with masonry wall below.)

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7.6: Girt Design and Damage Issues

In façade systems consisting of curtain walls or cladding elements – such as prefabricated panels, metal sidings, or similar assemblies (see for example Figure 7.14a) – the backup framing alone may not provide adequate resistance to lateral loads, particularly wind pressure, and cause excessive deformation and damage to the façade (see Figure 7.14b). This limitation is most pronounced when the cladding spans vertically, resulting in insufficient stiffness to carry wind loads without additional support. To address this, horizontal framing members known as "girts" are introduced. Girts function as intermediate horizontal beams that collect wind loads from the cladding and transfer them to the primary supporting structure. Proper girt design ensures that wind-induced pressures are effectively distributed, thereby reducing excessive deflection or localized failure of the cladding system.

Figure 7.15 illustrates the typical placement of a girt relative to the façade and its supporting structure. For comparison, Figure 7.14b shows the detrimental effects of excessive wind pressure on inadequately supported studs, which can lead to stud bending and subsequent damage to the cladding.

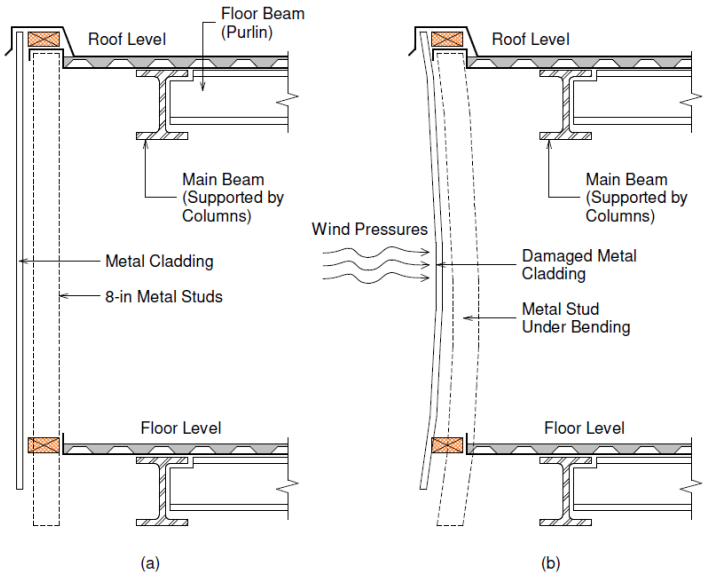


Figure 7.14 Significance of wind damage to façade (cladding) and supporting metal studs

(This figure shows two vertical cross-sections of a roof and a floor below. The floor beam cross-section at each level is shown providing support to metal studs and cladding. The detail on the left shows an undamaged metal stud and cladding, the one on the right depicts the effect of wind load causing the metal stud and cladding bend and damage.)

The situation depicted in Figure 7.14 is especially prevalent in façades where cladding can be subject to a heavy out-of-plane lateral load because of large exposure areas to wind. This can be, for example, in a building housing an industrial facility.

As indicated, Figure 7.15 shows the same detail, however, a girt has been added at the mid-height between the floor and the roof levels. As expected, girts are positioned in such a way to resist the bending in its strong axis. Girts are supported by the structural system and help shorten the bending span of the cladding, thus reducing the potential for excessive wind force effect on the cladding and preventing damage.

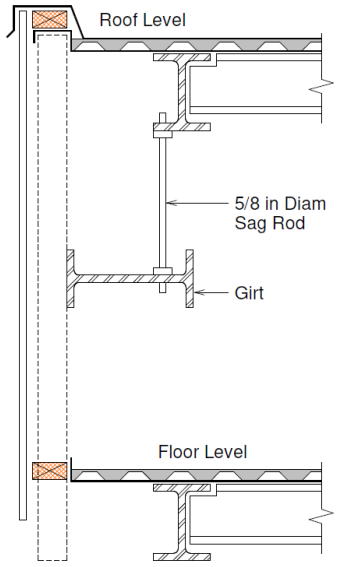


Figure 7.15 A girt is added at the mid-height between the floor and the roof to reduce wind load effect on the cladding

(This figure shows a vertical cross-section of a roof and a floor below. The floor beam cross-section at each level is shown providing support to metal studs and cladding. Furthermore, at the mid-height between the roof and the floor, the cross-section of a beam, known as a girt, is shown preventing the bending of the metal stud and cladding from wind loads. The movement of the girt in vertical direction is prevented using sag rods.)

It is noted that some claddings are able to support themselves; and as such, they do not need girts. Examples of these are reinforced concrete precast façade panels. When girts must be used, the following design considerations are important.

- As shown in the detail of Figure 7.15, sag rods are required to support the girts to prevent lateral buckling.
- Usually, two sag rods are used over the span length of the girt.
- Because a girt will be under bending in its weak axis supporting its own weight, in a way sag rods act as support for the girt – and as such, the girt behaves as a three-span continuous beam. In this way, its bending span will be reduced. This is illustrated in Figure 7.16.

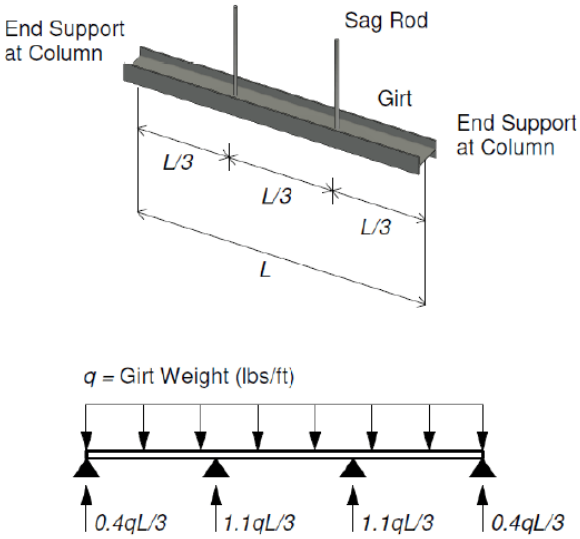


Figure 7.16 Idealization of a girt as a three-span continuous beam supporting its own weight

(This figure shows an isometric detail of a girt and its sag rods. The idealization of the girt as a continuous beam is also

shown with sag rods acting as intermediate supports carrying the self-weigh of the girt. The Intermediate reactions of the continuous beam are also shown.)

When designing a girt for the wind load, a simply-supported beam with a span length L is used. Against wind pressure, the girt will bend with respect to its strong axis of the cross-section. The load from wind is uniformly distributed on the girt with proper consideration for the tributary width of the wind pressure. However, as depicted in Figure 7.16, under its own weight, the girt is treated as a continuous beam with three spans. The intermediate reactions produce tension forces in sag rods. Thus the tension force in each sag rod, according to Figure 7.16, is equal to $1.1qL/3$, where q is the girt self-weight using for example pounds per foot (or kilonewton per meter) as the unit. However, it has been reported that because of the potential for buckling of the girt about its weak cross-sectional axis, a sag rod may actually experience a compression force. Although this compression force is somewhat compensated by the tension in the rod, the net result may still be a compression, which is critical for the sag rod design because of potential for buckling. This compression force, as suggested by Joseph Yura of the University of Texas (Stuart, 2020), is taken as 1% of the compression force that is developed in the flange of the girt as a result of the wind pressure. This compression force is compensated by the tension in the sag rod because of the weight of the girt. If the compression force is not fully compensated with the tension force, then a proper size for the sag rod must be selected such that the buckling strength of the rod is larger that the difference between the compression and the tension force.

Considering the bending moment in the beam due to the wind pressure as M, the compression force F developed in the flange is approximately equal to

$$F = M/d$$

where

$d = h - t_f$ with h being the beam depth and t_f the thickness of the flange.

Thus the net compression force in a sag rod (considering that there are two sag rods) is approximately equal to

Net compression force in a sag rod = $0.01F/2 - 1.1qL/3$

This compression force will need to be smaller than the buckling capacity of the sag rod.

Illustrative example – Example 7.8 in Appendix A demonstrates the design for a girt and its sag rod.

In the detail shown in Figure 7.15, a W-section is used and is positioned on its side. However, a channel (laid flat on its side), a hollow square section, or a C- or Z-profile are also considered as suitable structural members for girts.

In situations when the adjacent columns are far from each other, a larger steel section will be needed to support a girt. This makes the design very uneconomical and unreasonable. In these situations, it is more economical to add additional columns in the perimeter of the building to shorten the span length of girts. These additional columns are sometimes called wind (or soldier) columns. Also, if the main columns do not have adequate strength to carry the additional load transferred from the girt, wind columns are then used.

In seismic areas, consult with a local code to find out whether special provisions for design are required. However, most designers believe the design for wind is adequate enough for girts.

Signs of damage in girts - Signs of damage in girts are:

- Deformed metal siding; metal siding open at seams; connecting elements missing.
- Corrosion stains appearing on the cladding.
- Cracked or displaced cladding.
- Cracked and broken glass panes.

The repair, in most cases, requires that the façade and its elements (such as the cladding or the curtain wall) be removed to examine the condition of the girts. Necessary actions include the girt redesign and implementation of proper detailing including installation of sag rods.

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7.8: Exercises

E7.1 A lintel, spanning over an opening of L=10 feet (3.05 meters), uses an angle 6 by 6 by 9-16th of an inch thickness as its structural member. The lintel carries a load of a 9-foot-high (2.7-meter-high) brick wall above it. This is the only load. The brick wall masonry is 4-inch (102-millimeter) thick and is built in place, therefore, the interlock forces between the brick and mortar is present. The properties of the section are: S_x = 5.12 cubic inches (83,902 cubic millimeters) and I_x = 22.0 inches to the power of four (9.16 million millimeters to the power of four).

a) Compute the maximum bending moment M_u and the reaction R_u for this lintel.

b) Compute the nominal bending capacity of this section (M_n) and decide whether the section is acceptable. The unsupported length of the lintel is the same as its span length L.

c) Compute the maximum deflection of this lintel and decide whether the deflection is acceptable.

E7.2 Although popular in construction, it is generally a bad idea to use a flat plate (positioned horizontally) as a lintel in even relatively short spans. Select the answer that is the most plausible explanation for this.

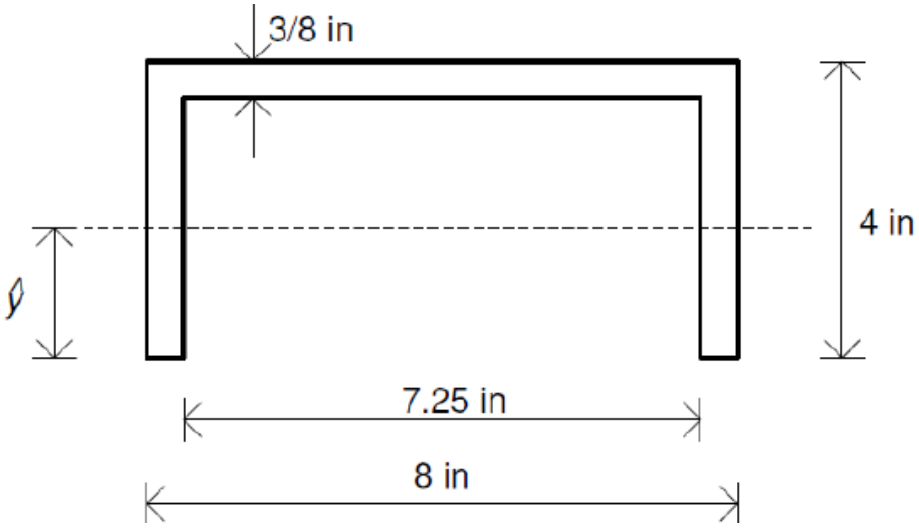
a) Flat plates are difficult to fit into masonry.

b) A flat plate of any reasonable width and thickness will not offer an economical solution because of small moment of inertia per weight ratio.

c) Flat plates of required sizes are difficult to find.

d) Flat plates have a smaller yield resistance compared with other structural profiles.

E7.3 While inspecting a façade and opening the area above a window, you noticed a profile as shown below (a U-section) is used as a lintel. The measurements shown were done at the site.



(This figure shows the cross-section of a channel-shape cross-section laid flat with the entire web dimension of 8 inches and width of each flange as 4 inches and 3-8th of an inch of thickness.)

a) Compute the location of the centroid of this section $\hat{y}$.

b) Compute the moment of inertia I_x.

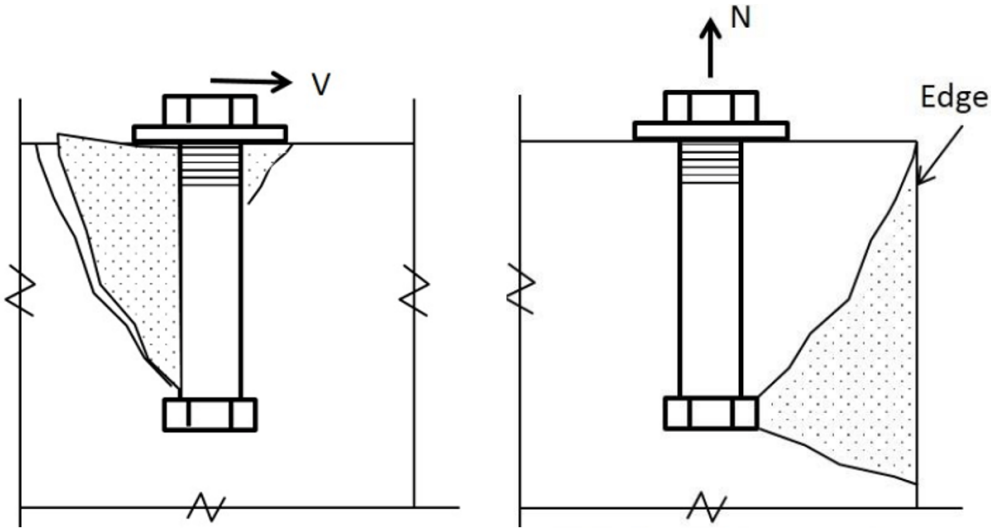
c) Compute the section modulus S_x.

d) It appears that this section was built by bending a flat plate of about 16 inches (0.4 meters) wide and 3/8 inches (9.5 millimeters) thick. If we used that flat plate laid horizontally, what would its I_x be?

e) Steel weighs about 490 pounds per cubic foot (7,849 kilograms per cubic meter). Compute the weight of the section shown above per linear foot (which is nearly the same as the weight of the flat plate per linear foot). Then compute the ratio of the $I - x$ to weight for both the U-section and the flat plate and decide which section is more efficient.

E7.4 In an effort to estimate the tension breakout strength of a type of anchor, several tests were conducted. The mean value and the standard deviation of the sample are 17.3 and 3.4 kips (77.0 and 15.1 kilonewtons), respectively. Based on this information, suggest a design value for the nominal strength of the anchor in kips.

E7.5 Examine the following figures and decide which modes of failure are corresponding to them.



(This figure shows two details. The one on the left shows a shear force on an anchor with a portion of the surrounding material separated behind the anchor. The one on the right shows an anchor, close to the edge, under a tensile force with a portion of the surrounding material separated at the edge.)

E7.6 In computing the strength of anchors embedded in concrete in seismic areas, a desirable condition is that the concrete-governed strength for a single anchor is greater than the steel strength of the anchor. Which of the following statements best describes the logic behind this requirement?

a) We want to allow steel yield first before the concrete is crushed.

b) We want both steel and concrete fail at the same time for a most economical design.

c) We want strong concrete to prevent the anchor pull-out.

d) We want the anchor survive even after the building is damaged.

E7.7 The following table summarizes the strength utilization ratios obtained for three tension and two shear modes of failure that were critical in design of an anchor embedded in concrete. Compute the interaction of tension and shear and decide whether the anchor is acceptable for design. Use the nonlinear interaction equation.

Mode of failure and Strength Utilization Ratio

Mode of Failure	Strength Utilization Ratio
Steel element tension failure	0.23

Concrete Breakout in Tension	0.49
Tension Pullout	039
Steel Element Shear	0.11
Concrete Pryout in Shear	0.58

E7.8 Which of the following statements is *incorrect*?

a) There is sometimes a possibility for anchor group failure in concrete breakout failure in tension.

b) The strength utilization ratio, when using the strength method, is the ratio of the factored strength to the factored load.

c) The reason the interaction equation for shear and tension is used is because the shear and tension work in two perpendicular directions.

d) The edge distance is an important factor in computing the concrete breakout strength in shear.

E7.9 In the Example 7.1 in Appendix A, and the shelf angle attachment to the concrete structure, we wish to use the same anchor, however with only 3 inches (76 millimeters) of embedment length. All other parameters are unchanged. Considering only two modes of failure: (1) concrete breakout in tension; and (2) concrete breakouts in shear:

a) Compute the factored strength value for concrete breakout in tension and the corresponding strength utilization ratio.

b) Compute the factored strength for concrete breakout in shear and the corresponding strength utilization ratio.

c) Use the nonlinear interaction equation and decide whether the anchor is ok, if we only consider these two modes of failure.

E7.10 An angle 5-inch by 5-inch and a quarter of an inch thickness is used as a shelf angle carrying the load of a 11-foot-tall (3.4-meter-tall) veneer wall of 4 inches (102 millimeters) in thickness. The gap between the veneer and the back-up wall is 1.75 inches (44 millimeters). The anchors are spaced at every 5 feet (1.52 meters) from each other. Ignore the self-weight of the angle.

a) Compute the factored tension force in the anchor.

b) Compute the factored shear force in the anchor.

E7.11 Which of the following statements is *incorrect*?

a) The steel element tensile strength of the anchor is independent of its embedment length.

b) A larger strength of the anchor for concrete breakout in tension is achieved when the edge distance is larger than 1.5 time the embedment length.

c) The group effect in concrete breakout in tension is sometimes involved in strength computation.

d) The group effect in concrete breakout in tension is always involved in strength computation.

E7.12 A cast-in place anchor with a bent end (hooked anchor) is used in masonry. The embedment length is 6 inches (152 millimeters). The nominal diameter of the anchor is 5-8th of an inch (16 millimeters), while the effective diameter is 1-8th of an inch (3 millimeters) less. The specified tensile strength of anchor steel is 58,000 pounds per square inch (399.9 megapascals) and specified yield strength of anchor steel is 36,000 pounds per square inch (248.2 megapascals). The material is ductile. Use

the LRFD method in this problem.

a) Compute the factored strength of this anchor in tension considering the steel tension failure.

b) Compute the factored strength of this anchor in shear considering the steel shear failure.

E7.13 Which of the following statements is *incorrect*?

a) The resistance reduction factor is usually larger for ductile materials than is for brittle materials.

b) The main idea in seismic design is to maintain adequate ductility.

c) The strength values of an anchors for all modes of failure are always the same regardless of whether we are designing in a seismic area or not.

d) The basis of the ACI's strength method is in introducing load and resistance factors similar to the LRFD method.

E7.14 An angle 5-inch by 5-inch and one half an inch thickness is used as a shelf angle to support a 4-inch (102-millimeter) brick veneer wall of 7.5 feet (2.3 meters) in height. The anchors are installed at every 4 feet (1.2 meters) spacing attaching the shelf angle to a grouted CMU masonry wall of adequate design to accommodate the anchors. The anchor edge distance is 10 inches (254 millimeters). The cavity between the CMU wall and the brick veneer is 2 inches (51 millimeters). The anchor used has the following properties: cast in-place headed A307 type with a diameter equal to 5-8th of an inch (16 millimeters); the embedment length is 4.5 inches (114 millimeters). Note: the yield resistance of the anchor steel is 36,000 pounds per square inch (248.2 megapascals). The masonry compressive strength f'_m = 1,500 pounds per square inch (10.3 megapascals). Use ASD method in this problem.

a) Compute the axial load and shear in the anchor (neglecting the self-weight of the angle and using the lever arm as 3 – $t/2$ inches or 76 – $t/2$ millimeters).

b) Compute the ASD resistances for tension and shear for the anchor.

c) Using the interaction equation for tension and shear, decide whether the anchor is acceptable.

E7.15 Assume the anchor selected in E7.15 is not adequate for the angle. Which of the following actions is then most effective to make the design acceptable?

a) Using a larger bolt diameter.

b) Using a spacing S = 3 feet (0.9 meters) instead of the 4 feet (1.2 meters).

c) Using an anchor with a larger yield resistance.

d) Using a larger size angle.

E7.16 An angle 8-inch by 8-inch by 1 inch thickness is suggested for a shelf angle which is attached to a structural steel frame system. The Angle spans over 14 feet (i.e., L = 14 feet or 4.3 meters). The lateral restraints are at ends only. The wall supported by the angle is a 4-inch (102-millimeter) brick with a height of 12.6 feet (3.8 meters). Use the LRFD method in this problem. The section properties for the angle are S_x = 15.8 cubic inches (258,916 cubic millimeters), I_x = 89.1 inches to the power of four (37.09 million millimeters to the power of four). For this angle, F_y = 36,000 pounds per square inch (248.2 megapascals).

a) Compute the factored moment M_u and end reaction R_u of this angle.

b) Compute the nominal moment resistance M_n of the angle.

c) Compute the angle deflection in inches.

d) Based on your findings in (a), (b) and (c), decide whether the angle design is satisfactory.

e) The connection of this angle to the structural steel is via two A325 bolts. However, for the welded part, a 4-inch (102-millimeter) strip of weld of 3-16th of an inch (5 millimeters) in size is suggested for the connection of this angle to

the structural system (via a plate bracket). The eccentricity of the reaction R_u with respect to the weld is also 4 inches (102 millimeters) and the electrode type is E70. Compute the factored shear resistance of the weld using the AISC tables. Based on your finding and the R_u you computed in Part (a), decide whether the weld is satisfactory.

E7.17 Girts refer to beams that support claddings in facades against wind loads. We often need two sag rods installed along the span length of a girt. Why do we need these sag rods?

a) Sag rods are needed to attach cladding to the interior structure.

b) Sag rods are needed to reduce the applied wind pressure.

c) Sag rods are needed to prevent lateral buckling in the girt.

d) Sag rods are needed for temperature control.

E7.18 in design for a sag rod, we must make sure the compression force developed in the sag rod does not exceed the buckling resistance of the rod. A sag rod has a length of 14.5 feet (4.4 meters) and 3 quarters of an inch (19 millimeters) diameter. Compute the value for the buckling strength, F_e (note: for a circular cross section, the radius of gyration is 1-4th of the diameter).

E7.19 Which of the following statements is not accurate in regard to girt design?

a) A girt is designed as a beam resisting wind forces transferred from the claddings.

b) The most critical load for a girt design is its self-weight.

c) The compression force in a sag rod for girts is suggested to be taken about 1% of the compression developed in the flange of the girt.

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8: NON-DESTRUCTIVE EVALUATION AND FAÇADE MONITORING FOR DISTRESS CONDITIONS

Chapter Synopsis

This chapter presents an overview of non-destructive evaluation (NDE) methods used in the investigation and assessment of building envelopes. NDE encompasses a range of non-invasive techniques designed to evaluate the condition of materials and structural components without impairing their serviceability. These methods include in-situ testing, long-term monitoring of damage progression, localized core sampling for calibration or verification, and laboratory analyses of samples taken from façades.

Accurate diagnosis of façade deterioration or distress often requires combining multiple NDE approaches to determine both the underlying causes and the extent of damage. This chapter discusses the principles, applications, advantages, and limitations of the major NDE techniques relevant to masonry, concrete, metal, and composite cladding systems. Emphasis is placed on their role in supporting critical examinations of existing building envelopes, informing maintenance strategies, and guiding repair or rehabilitation decisions.

8.1: Introductory Remarks

8.2: Common NDE and Monitoring Methods for Façade Inspections

8.3: Need for NDE Methods and Monitoring

8.4: Consultant Services Needed for NDE Methods and Monitoring

8.5: Procedures and/or analyses used in several common NDE methods

8.6: Vibration Analysis

8.6.1: Computing fundamental natural frequency of vibration of beams

8.6.2: Retrofitting a beam or lintel to control vibration

8.7: Chapter References

8.8: Exercises

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8.1: Introductory Remarks

A non-destructive evaluation (NDE) encompasses all investigative activities intended to assess the condition of a façade through testing and analysis, whether conducted in the laboratory or in the field. NDE includes all forms of non-destructive testing (NDT) that may be required as part of the inspection process to characterize material properties, detect flaws, or evaluate structural integrity without causing damage to the façade or its components.

Monitoring, on the other hand, refers to the systematic observation and measurement of changes occurring over time in a façade system exhibiting distress or deterioration. Monitoring techniques are particularly valuable in identifying the progression of cracking, displacement, or moisture migration and in verifying the effectiveness of remedial measures.

In practice, both NDE and monitoring are often employed concurrently to determine, with reasonable accuracy, the cause, mechanism, and extent of façade distress, thereby enabling the selection of appropriate repair or rehabilitation strategies. The choice of specific NDE techniques depends on several factors, including:

- The extent and severity of the observed damage.
- The degree of uncertainty associated with the distress mechanism.
- The structural behavior and material characteristics of the façade or its components.
- The availability and reliability of existing documentation, such as as-built drawings, prior inspection reports, or engineering analyses.

Recommendations for implementing NDE or monitoring are typically formulated after completion of the visual inspection and prior to the initiation of repair or rehabilitation work to ensure that all subsequent interventions are based on a sound diagnostic understanding of the façade condition.

In façade condition assessments, non-destructive evaluation (NDE) methods are employed to obtain critical diagnostic information about the behavior, performance, and integrity of façade systems without impairing their function. The primary objectives of NDE include:

- Determining the mechanical and physical properties of façade materials.
- Investigating conditions in concealed or inaccessible areas.
- Identifying the presence of potentially hazardous conditions.
- Determining the underlying causes of observed distress or deterioration.
- Assessing deficiencies in the structural integrity of wall assemblies and their supporting components.

For example, NDE techniques may be used to locate reinforcement, embedded anchors, or voids within concrete or masonry walls, or to detect delamination, cracks, or corrosion that are not visible on the surface. Such evaluations provide essential data for diagnosing façade performance issues and for developing informed maintenance, repair, or rehabilitation strategies.

- Monitoring activities are often used in identifying the cause of a distress condition in situations when the particular condition is changing with time. And as such, observations and measurements over some period of time become necessary. Monitoring activities in façade condition assessment are often conducted for the following situations:
- Measuring crack growth in masonry and concrete walls
- Measuring vibrations in the vicinity of such structural elements as lintels, floor and roof joists, and portions of walls
- Observation and measurement of moisture changes and water infiltration in, and through, façades
- Wall movements because of soil pressure
- Water table measurements, when ground water level is targeted as a cause of distress in a wall and its foundation
- Temperature variations and heat distribution on façades
- Acoustic measurement

Some of the materials covered in this chapter can also be found in Mohammadi et al. (2022).

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8.2: Common NDE and Monitoring Methods for Façade Inspections

Distress conditions in façades are often unique; and as such, it will be difficult to come up with a series of generic methods to apply equally to all cases. Several more common NDE methods, that are often used, and their applications are summarized in Table 8.1. A few common monitoring methods used in façade inspection and condition assessment are summarized in Table 8.2. Certain monitoring devices can be preprogrammed and left at the site for automatic reading and recording changes in parameters that describe a particular distress parameter. However, in many applications, periodic measurements will need to be performed and manually recorded.

Table 8.1 Common NDE methods (Mohammadi et al., 2022)

Method	Description	
Acoustic measurements	Measurement-grade microphone is used to measure the intensity of sound	In situations when excessive occupants' comfort
Asbestos investigation	Removing samples in areas where asbestos is suspected	Determination of extent of as rehabilitation is planned)
Concrete cover removal	Concrete cover in small areas is removed to expose rebars in concrete walls	Determination of condition o damage due to spalling
Core sampling	A sample of specific size is removed from concrete wall for conducting lab tests	Determination of concrete m including identification of ag
Coupon sampling	Small sample size of material is removed for conducting lab tests	Determination of mechanical
Environmental tests	Taking samples of paint and molds for lab tests	Lead paint contents and mol
Ground Penetrating Radar (GPR)	GPR device uses electromagnetic waves and detects the reflected signals from subsurface anomalies	Concrete subsurface void de amount of concrete cover
Laser Imaging, Detection, and Ranging (LIDAR)	Laser equipment and special camera are used	Distance measurement to ide
Moisture measurement or probe (e.g., in Exterior Insulation Finishing System, EIFS)	Using moisture measurement device	Detection of moisture within efflorescence situations

Exploratory openings made in façade	One or more wall components or cladding openings (bricks, concrete masonry unit, terra-cotta, stones, metal panels, glass, cladding sections are removed)	Investigating the condition b metal rods that secure stone Barrier (WRB), concealed sea
Soil explorations	Removing soil around foundation to expose it for inspection	Condition assessment of fou suspected in causing cracks
Thermography	Special cameras are used to take images of heat distribution on wall surface	Thermal images are used in possible poor insulation or w
Water spray test	Spraying water on windows and walls following a specific pattern over a sufficient length of time	Leak detection especially at windows and wall penetratior

Table 8.2 Common monitoring methods (Mohammadi et al., 2022)

Method	Description	
Acoustic monitoring	Measurement-grade microphone is used for periodic reading of intensity of sound	(a) Measuring variation ir (b) Used with an array of vibrations that can be tri
Air monitoring	Air samples are measured at intervals	Detecting environmental
Crack monitoring	Crack meter is installed at the crack on the wall	Measuring change in cra
Moisture monitoring	(a) Probes are used to measure moisture (example in EIFS) or areas behind a wall (b) Probes are used to measure temperature and relative humidity at building interior spaces, metal wall components, and wall cavities	(a) Monitor moisture vari (b) Monitor surface and a dew point concerns
Vibration monitoring	Accelerometers are used for measuring vibration amplitudes with time	Identifying frequency cor
Wall movement	(a) Camera installed afar aiming at targets installed on wall (b) Tilt meters installed on walls to monitor movement	(a) Movement of wall by t (b) Inward or outward wa
Water table	Rod is used in wells dug near the wall to monitor water table	Water table variation with suspected of foundation

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8.3: Need for NDE Methods and Monitoring

Visual investigations are often supplemented with non-destructive evaluation (NDE) techniques to obtain a more comprehensive understanding of the nature, extent, and causes of façade distress. For example, when assessing damage associated with lintels, shelf angles, or terra-cotta units, it is often necessary to gain access to concealed areas behind the façade. This may be accomplished by creating limited exploratory openings to directly observe concealed conditions and verify the integrity of supporting elements.

Openings may also be required when anchor deficiencies are suspected – such as missing, corroded, or failed anchors that support veneer wall systems. In façade investigations, such localized exploratory openings are generally considered routine NDE procedures that complement visual inspection. However, in cases involving complex or progressive deterioration, more advanced NDE techniques or long-term monitoring may be necessary to accurately characterize structural behavior and guide repair or rehabilitation strategies.

When the façade inspection professional determines that non-destructive evaluation (NDE) or monitoring is warranted, it is essential to document the findings and rationale for such recommendations. The owner should be provided with a clear description of the proposed method, the purpose and justification for its use, the anticipated cost, and, where applicable, the duration and frequency of monitoring activities. The documentation should also outline the expected range of possible outcomes and how these results may influence subsequent repair or rehabilitation decisions.

As the architect or engineer responsible for the façade investigation, the professional should proactively discuss with the owner the potential need for NDE and/or long-term monitoring. To ensure effective communication and manage client expectations, the possibility of supplemental testing or monitoring should be conveyed before the commencement of visual inspection activities. This approach facilitates informed decision-making, budget planning, and alignment of project scope with the investigative objectives.

In preparation for implementing NDE, the professional must exercise due diligence and include the following in their proposal to the owner:

- A statement reserving the right to recommend additional testing and/or monitoring if necessary – especially in situations where visual inspections alone cannot adequately assess the extent of distress or when hazardous materials (e.g., asbestos, lead paint, or mold) are suspected.
- A disclosure that additional tests and/or monitoring may require engaging specialized experts or licensed professionals, which could incur additional costs.

While NDE methods often involve compiling data in one test or observation setting, monitoring involves collecting data over time. Monitoring is generally necessary in the following situations:

- The issue causing façade distress or damage is persistent rather than temporary.
- A single inspection or sampling does not adequately represent the typical distress conditions observed.
- External changes, such as nearby construction or alterations to the building's loading regime, are occurring.

It is necessary to make sure that conditions contributing to the distress have been stabilized (for example, foundation settlement has ceased) prior to implementing remedial actions.

Once the data are compiled, the professional is able to conduct the necessary analysis, and complete the evaluation report with more certainty about the causes of distress and condition of the façade.

Figure 8.1 shows the flow of activities leading to the task of façade condition assessment and completion of an evaluation report.

In performing the preliminary investigation of the condition of the façades and exercising the due diligence, special attention may be needed for conditions that indicate need for NDE methods or some type of monitoring. Table 8.3 provides a summary of several such situations and the type of NDE methods and/or monitoring that may be needed.

Engineering evaluation and analysis form a critical component of façade condition assessment when employing non-destructive evaluation (NDE) methods and monitoring techniques. Depending on the type and complexity of the observed distress, the evaluation may range from qualitative visual interpretation to quantitative engineering analysis supported by detailed

calculations.

For instance, in some cases, evaluation may be limited to direct observation – such as when localized openings are created in the wall to assess the condition and integrity of anchors, ties, or concealed wall components. In other cases, more advanced analytical procedures are required. An example is the investigation of vibration-related distress in structural members such as lintels, where acceleration data are collected to study dynamic response characteristics. Such analyses may include evaluating the frequency content of the induced vibrations, assessing the potential for excessive oscillation, and identifying resonance conditions that could result from the operation of nearby mechanical equipment (e.g., air-conditioning units transmitting vibrations to the lintel).

Through this combination of observation, measurement, and engineering computation, a more accurate understanding of the façade's structural behavior and the root causes of distress can be achieved.

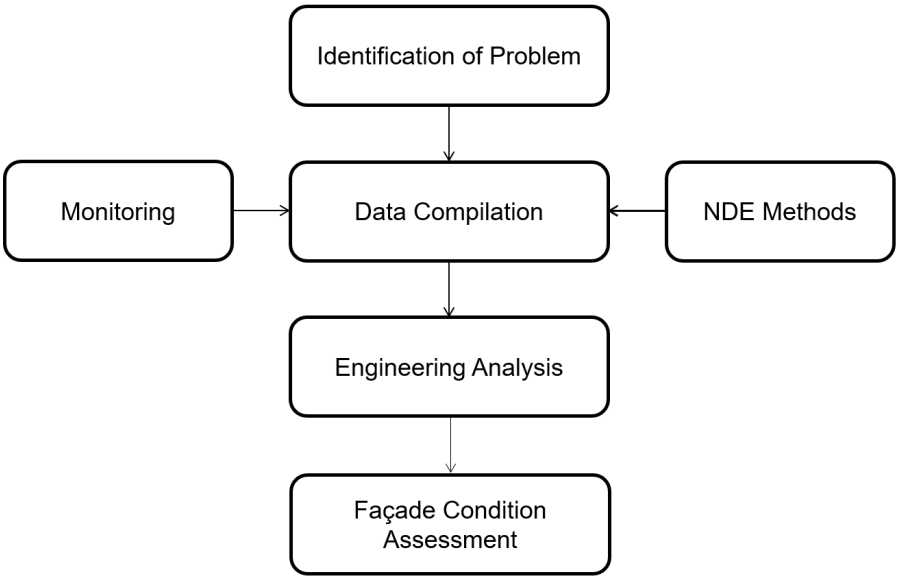


Figure 8.1 Flow of activities in compiling NDE and monitoring data for façade condition assessment (Adapted from Mohammadi et al., 2022)

(This figure shows a flowchart; at the top the statement says identification of problem. The following steps below that indicates data compilation, engineering analysis and façade condition assessment.)

Table 8.3 Types of conditions that may necessitate NDE methods and monitoring (Mohammadi et al., 2022)

Situation	Reason for NDE methods or monitoring	Possible ty
Old buildings with lack of maintenance and repair	Lack of documentation from previous inspection; non-existent structural and architectural drawings; multiple factors causing distress conditions	Coupon samplin crack monitorin
Severe deformation of veneer walls (bulged out wall)	Lack of, or damage to, supporting anchors suspected	Exploratory oper
Unexplained large cracks especially on first floor walls	Foundation settling causing cracks in façade; a crack is expected to change with time	Crack monitorin related to the fo

Complaints by occupants regarding moisture in their units when rains	Flashing above and below windows are damaged or missing	Water spray test
Severe rust stains present on facades	Damaged lintels, shelf angles, or facade connections	Exploratory oper
Displaced, damaged, or cracked parapet walls	Corroded shelf angles; steady vibration present; missing or damaged flashing	Vibration monito
Large openings between window frames and surrounding wall	Wall movement (because of foundation settling)	Wall movement
Almost all facades made up of terra cotta blocks	Corrosion or damaged attachment rod	Exploratory oper
Bricks severely damaged at locations of iron fire escapes attachment	Deteriorated or corroded anchor or bracket attaching stair to wall	Exploratory oper
Old lead-based paint suspected (prior to 1978)	Hazardous condition for any repair, rehabilitation, or reconstruction work	Environmental te
Asbestos fibers, tiles, panels, etc. suspected	Hazardous condition for any repair, rehabilitation, or reconstruction work	Environmental te
Efflorescence deposits present	Determining composition and cause	Samples taken t moisture measu
Buildings subject to vibration	Vibration in structural members causing distress and cracks in façade and/or occupant complaints	Vibration monito
Complaints by occupants regarding excessive noise	Understanding the level of noise	Sound level mea
Complaints by occupants of condensation/ frost formation on interior surfaces	Existence of thermal bridges; inadequate interior heat sources or high levels of interior relative humidity; poor air circulation	Wall openings; t conduction heat modeling
Glass damage or breakage	Spontaneous or thermal breakage and glass movement within frames	Glass removal; g Microscope (SEI

As illustrated in Figure 8.1, engineering evaluation and analysis represent a key component of façade condition assessment when non-destructive evaluation (NDE) methods and monitoring are applied. The scope of this evaluation varies according to the nature and severity of the distress condition. In some cases, the assessment may be primarily observational, involving targeted openings in wall assemblies to directly inspect and verify the condition and performance of concealed anchors or ties. In other cases, the evaluation requires quantitative analysis supported by detailed engineering calculations and data interpretation.

For example, when investigating potential vibration-related distress in a structural element such as a lintel, acceleration measurements may be recorded and analyzed to determine the vibration frequency spectrum, assess the risk of excessive movement, and identify possible resonance effects induced by adjacent mechanical equipment or dynamic loads.

This structured approach – integrating field observation with analytical evaluation – ensures that the façade's structural behavior is accurately characterized, providing a reliable basis for diagnosing causes of distress and developing appropriate remedial strategies.

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8.4: Consultant Services Needed for NDE Methods and Monitoring

Services of third-party consultants are often needed when NDE and monitoring become necessary. Several types of expertise, including glass façade and fire expertise, that are often hired for these services are outlined below. It is noted that because of the uniqueness of the glass problems and issues related to fire effects, more detailed information, under these two topics, are provided to familiarize readers with the types of problems that glass walls and fire issues may impose on façades.

Acoustics expert services – In certain situations, measuring ambient noise levels and monitoring their variation over time becomes an essential component of a comprehensive façade or building envelope evaluation. This is particularly important when external noise sources – such as mechanical equipment, vehicular traffic, or adjacent construction activities – have been identified as potential contributors to occupant discomfort or building performance issues. In such cases, the involvement of a qualified acoustic engineer is necessary to perform detailed sound level measurements, conduct long-term acoustic monitoring, and analyze the frequency characteristics of the noise. Based on these findings, the acoustic specialist can recommend appropriate noise mitigation and control measures, which may include architectural modifications, installation of sound barriers, or improvements in the façade's sound transmission class (STC) and outdoor-indoor transmission class (OITC) performance.

Asbestos services – Asbestos can be a highly hazardous material; it will need to be investigated and abated by experts called "asbestos consultants." These are licensed professionals who is able to

- Inspect or evaluate a building for asbestos hazards.
- Develop asbestos abatement plans.
- Conduct site inspections.
- Perform air monitoring.
- Follow provisions of industrial hygiene services related to asbestos abatement.
- Provide supervision on all activities related to asbestos removal or abatement.

Depending on the local municipality and pertinent codes regarding public health, more than one kind of asbestos consultant may be required in a project. In general, the asbestos consultant's expertise may be in several areas as summarized in Table 8.4.

Table 8.4 Types of asbestos consultants (DPH, 2015)

Title designation	Description and expertise
Inspector	A licensed asbestos consultant certified as an inspector who identifies, assesses the co samples of suspected asbestos-containing material
Management planner	A licensed asbestos consultant certified as a management planner who uses data gathe to assess asbestos hazards, determine responses, and develop implementation plans
Project designer	Any licensed asbestos consultant certified as a project designer who determines how as be conducted and prepares the plans, designs, procedures, work-scope, or other substa abatement projects
Project Monitor	Any licensed asbestos consultant certified as a project monitor who functions as an on- facility owner or manager, by overseeing the activities of the asbestos abatement contra

EIFS evaluation and inspection services – Exterior Insulation and Finish System (EIFS) is a synthetic stucco-type finish applied

on the façade exteriors. Distress conditions in façades made up of EIFS often involve water intrusion problems that are not apparent from a visual inspection of the exterior. Special NDE methods and/or monitoring services may be necessary, and need to be done by EIFS experts, to identify the extent of moisture build-up behind the finish, potential damage to the wood sheathing and boards, and provide recommendations for repair.

Environmental services for mold situations and presence of lead paint – Areas suspected of mold growth or lead-based paint contamination must be inspected by qualified environmental engineers or industrial hygienists. These professionals are responsible for conducting a comprehensive environmental assessment to determine the presence, type, and extent of contamination. The investigation typically includes the collection of surface and air samples for laboratory analysis to identify mold species or quantify lead concentrations. Based on the analytical results, the environmental specialists develop abatement and remediation recommendations consistent with applicable environmental health and safety standards, such as those established by the U.S. Environmental Protection Agency (EPA) and the Occupational Safety and Health Administration (OSHA). These recommendations may address both immediate corrective measures and long-term prevention strategies to mitigate future contamination or exposure risks.

Fire protection expertise – Building façades – such as curtain walls, veneers, and other cladding systems – are often subject to fire protection and life-safety requirements established by building and fire codes. When a façade system exhibits distress conditions that compromise its ability to perform as part of the building's fire protection assembly, the involvement of a qualified fire protection engineer with specialized expertise becomes essential. Evaluation of façade performance with respect to fire resistance and endurance must be conducted in compliance with the applicable provisions of national and local building codes, as well as standards issued by organizations such as NFPA (National Fire Protection Association) and ASTM International. The assessment typically focuses on determining the façade's ability to limit fire and smoke propagation and maintain its integrity under elevated temperature exposure.

Containment of fire and smoke within the compartment of origin is critical to life safety and property protection. Modern buildings achieve this through a combination of active fire suppression systems (e.g., sprinklers) and passive fire protection measures such as fire-rated walls, floors, and perimeter barriers. These fire separations are designed to confine the fire to its originating area or, at minimum, prevent vertical or horizontal spread beyond the floor of origin. However, exterior wall systems, including curtain walls and certain veneer assemblies, are often not required to have fire-resistance-rated construction. This can render them more vulnerable to external fire exposure and potential failure at the perimeter slab-to-wall interface. Fire and smoke can propagate through unsealed or inadequately protected perimeter joints between the floor slab edge and the curtain wall, allowing flames and hot gases to breach upper floors – even when the floor assemblies themselves meet fire-resistance requirements.

Several high-rise building fire incidents during the 1980s and 1990s demonstrated this vertical fire spread mechanism, leading to enhanced code provisions in the United States. Current building codes now mandate fire-resistance-rated perimeter joint systems, designed and tested to prevent fire and smoke transmission through these vulnerable interfaces, thereby improving the overall fire performance of the building envelope.

Another mechanism for vertical fire spread in buildings involves the use of combustible materials within curtain wall assemblies, such as plastic foam insulation, composite cladding panels, or other polymer-based components. Once ignited, these materials can sustain and propagate flame and heat both vertically and horizontally across the building's exterior, leading to rapid fire spread over multiple stories.

One of the most tragic and well-documented examples of this phenomenon was the 2017 Grenfell Tower fire in London (LFB, 2019), in which combustible cladding materials significantly contributed to the rapid upward and lateral spread of flames, resulting in severe loss of life and extensive structural damage. Similar incidents have occurred in other high-rise buildings worldwide, reinforcing the critical importance of fire-safe façade design and material selection. In recognition of such hazards, U.S. building codes since 2000 have incorporated stringent requirements for non-load-bearing exterior wall assemblies that contain combustible materials. Specifically, curtain wall systems used in buildings exceeding a prescribed height must successfully pass the NFPA 285 fire test (NFPA, 2019). This full-scale, two-story test method evaluates the vertical and lateral flame spread, heat transmission, and structural integrity of an exterior wall assembly when exposed to a standardized fire source. Compliance with NFPA 285 demonstrates that the assembly can resist the spread of flames and hot gases beyond the area of origin, satisfying both performance and safety criteria.

Through appropriate design and testing, curtain wall systems incorporating fire-retardant-treated combustible components or effective fire blocking and thermal barriers can achieve acceptable fire performance while maintaining desired architectural and energy-efficiency attributes. Given the critical nature of these fire-safety considerations, it is essential that the architects and engineers responsible for façade condition assessments possess a thorough understanding of fire protection design principles, test standards, and applicable code provisions. The professional in charge must be prepared to engage specialized fire protection

engineers when needed, to ensure that façade systems under investigation are evaluated and maintained in full compliance with modern fire-safety requirements.

Glass façade expert - Glass usage in facades is becoming more extensive, especially in office and commercial buildings, in the form of window panes, window walls, and curtain walls. Because of its special features, façades containing glass, whether as window panels or curtain walls, require special expertise resulting in the hiring an expert to evaluate potential problems. In cases of windows and glass panels framed into façades, major problems that require attention include water leakage, obtrusive visual anomalies or distortions, thermal issues, installation deficiencies, and glass breakage.

The following common façade problems are associated with window walls and curtain walls and constitute important information that the professional needs to know when hiring a consultant with proper expertise.

Moisture penetration – In situations where the glass installation consists of double pane or Insulating Glass Units (IGUs), the edge seal (a component of the edge spacer system consisting of the perimeter components separating the two panes of glass) may fail allowing moisture to enter the sealed air space. This may result in discoloration, fogging, condensation, and even ice formation between the glass panes. The glass consultant can provide NDE services consisting of an IGU design review including age of the IGUs, IGU removal to examine installation, visual examination of conditions, and perform frost point testing when assessing distress.

Spontaneous breakage – This type of spontaneous fracture phenomenon does not occur in annealed glass but is specific to fully tempered glass. It results from the expansion of nickel sulfide (NiS) inclusions that become embedded within the glass matrix during manufacturing. During the tempering process, glass is rapidly cooled (quenched) from a high temperature, which induces compressive surface stresses and tensile core stresses. However, this rapid quenching prevents nickel sulfide inclusions from completing their high-temperature to low-temperature phase transformation. Over time, and particularly with subsequent thermal exposure after installation, these nickel sulfide inclusions may gradually undergo phase transformation and volumetric expansion. Depending on the size, distribution, and location of the inclusions within the glass matrix, the resulting localized internal stress can exceed the tensile capacity of the tempered glass, ultimately leading to spontaneous fracture. Such failures may occur months or even years after fabrication, long after the façade has been installed (Barry and Ford, 2001; Karlsson, 2017).

Recognizing the risks associated with nickel sulfide contamination, glass manufacturers have implemented quality control and mitigation measures. These include reducing the introduction of nickel-containing contaminants during production and employing heat-soak testing (HST) – a process in which tempered glass panels are reheated under controlled conditions to accelerate any latent NiS-induced fracture, thereby minimizing the probability of in-service breakage.

When spontaneous glass breakage is observed, a qualified glass consultant can perform non-destructive evaluation (NDE) and forensic analyses. Typical services include reviewing the history and frequency of glass breakage, analyzing fracture patterns to identify failure origins, extracting and testing materials from the fracture initiation site, and evaluating any special features of the glass composition or lamination that may contribute to breakage or leakage.

Thermal stress breakage – Thermally-induced stresses can develop in glass panels when temperature differentials occur across the surface of the glass due to non-uniform solar radiation exposure. Typically, the central portion of the glass becomes warmer than the edges, which remain relatively cooler due to contact with framing systems or shading. This temperature gradient can produce tensile stresses at the glass edges that may exceed the material's tensile strength, leading to thermal fracture.

- Several factors influence the magnitude and distribution of thermal stresses in glazing systems, including:
- Exterior shading conditions and surrounding obstructions.
- Glass type (e.g., clear, tinted, or reflective).
- Coating type, emissivity, and surface location.
- Use of insulating glass units (IGUs) within the façade system, including spandrel and shadow box applications.
- Façade orientation and exposure conditions.

Because thermal breakage typically initiates at the glass edges, the edge condition and quality play a critical role in performance. Edge strength can be significantly reduced by pre-existing flaws, such as microcracks, chips, or handling damage introduced during fabrication, transport, or installation.

In cases where thermal breakage has occurred or is suspected, the services of a qualified glass consultant are recommended. Such professionals can provide non-destructive evaluation (NDE) and forensic investigation, including:

- Reviewing glass breakage history and frequency.

- Examining edge conditions and fracture patterns to identify the failure origin.
- Assessing glass coatings, heat treatment, and manufacturing processes.
- Evaluating façade exposure and shading conditions.
- Performing thermal stress analyses to estimate critical temperature differentials and assess risk of recurrence.

These evaluations assist in identifying the root cause of thermal failure and in developing effective mitigation and design strategies for repair and future installations.

Water leakage – Water may bypass the weathering seals around the glazing perimeter and contribute to edge seal failures (fogging) and interior water leakage depending on the water management approach of the glazing system and installation conditions including drainage capabilities and glazing seals. Water intrusion can contribute to organic growth on interior finishes and cause wall component distress.

As expected, the variety and uniqueness of these stress conditions require services of an expert who can identify the distress condition, pinpoint the cause and prescribe any necessary NDE and/or monitoring methods as needed.

Soil explorations and engineering services – When foundation problems are suspected of causing façade distress conditions, such as wall movement, cracks, and window frame distortion, these services are hired for their expertise in soil mechanics, foundation design, foundation settlement, and foundation stabilization.

Vibration measurement services – Vibration monitoring requires installing accelerometers in the affected areas and measuring the level of acceleration and its changes with time. Furthermore, the data compiled will need to be analyzed to provide information on the characteristics of the input vibration and its potential mitigation. Thus, experts will need to be hired to provide these services.

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8.5: Procedures and/or analyses used in several common NDE methods

In this section, an overview of the procedure and/or analyses used in conjunction with several NDE or monitoring methods is presented. Vibration analyses are covered separately in Section 8.6.

Crack monitoring – Crack monitoring is a diagnostic method used to determine whether an existing crack in a façade or structural element is stable or actively propagating. This process is critical in assessing the progression of distress conditions and in determining the appropriate timing and extent of repair interventions. One of the most common and cost-effective monitoring tools is the mechanical crack meter (ruler-type gauge). These gauges typically consist of two transparent plates that slide relative to one another, allowing small displacements across the crack to be visually measured. The plates are securely bonded to the façade surface on opposite sides of the crack using fast-setting epoxy adhesive. Once installed, readings are taken at regular intervals – often every few days or weeks – and the data are recorded to track any changes in crack width or displacement.

In addition to mechanical gauges, electronic or sensor-based crack monitoring devices are available. These systems can automatically record crack width variations over time and transmit the data digitally, allowing for continuous monitoring and more precise evaluation of crack activity. Although such devices are more expensive, they provide valuable long-term data, particularly for critical or difficult-to-access façade locations.

If progressive increases in crack width or length are observed, this indicates that the underlying causative mechanism remains active, necessitating further engineering evaluation. Conversely, if no measurable crack growth occurs over an extended monitoring period, it can be reasonably concluded that the condition has stabilized, and appropriate repair or restoration measures may be initiated.

Crack meter reading and interpretation – Figure 8.2 illustrates the typical configuration of a mechanical crack meter and the method of data interpretation. The device includes a reference crosshair, consisting of two bright red intersecting lines (one horizontal and one vertical), and a calibrated grid etched on the underlying plate. The intersection point of the red lines represents the zero-reference position, corresponding to the initial reading when the device is first installed.

As the crack widens or shifts, the two plates slide relative to each other, causing a visible displacement of the red reference lines against the black gridlines. The gridlines are spaced at 1-millimeter intervals, allowing direct measurement of relative movement in both horizontal and vertical directions.

Each observation records the movement of the red lines relative to the grid and is expressed as $H(x_1, x_2)$ for the horizontal direction and $V(y_1, y_2)$ for the vertical direction, where $H(x_1, x_2)$ indicates the displacement of the vertical red line as measured on the top and bottom horizontal scales of the grid; while $V(y_1, y_2)$ indicates the displacement of the horizontal red line as measured on the left and right vertical scales of the grid.

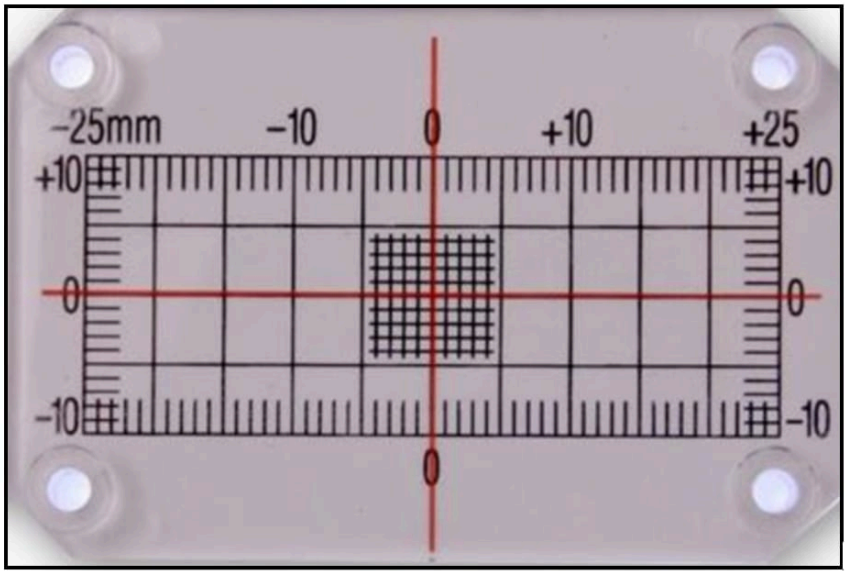
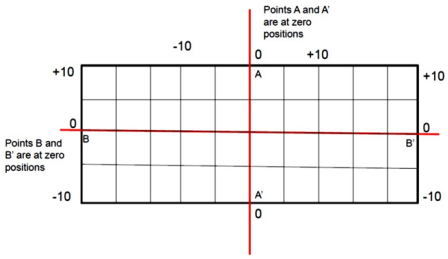


Figure 8.2 Gridlines (hash marks) on crack meter, adapted from Mohammadi et al. (2022)

(This figure is an image with gridlines situated vertically and horizontally creating a 5-millimeter by 5-millimeter gridlines.)

At the initial reading, H(0, 0) and V(0, 0) represent the original alignment, with no displacement detected. Subsequent readings reflect the relative shift of the red reference lines, allowing accurate quantification of crack movement in two orthogonal directions. As an example, let's assume upon initial installment of a crack meter, and after three months, we have a reading of H(1,-2) and V(3,-2). This indicates that the vertical solid red line at the top is moved 1 millimeter to the right, while the bottom part of it has moved 2 millimeters to left; and, similarly, the horizontal line is moved down by 2 millimeters on the right and moved up by 3 millimeters on the left. These changes are shown in Figure 8.3.



(a) Original position of AA′ and BB′ solid lines

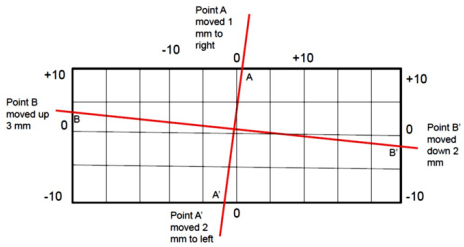


Figure 8.3 Examples of crack meter readings H(1,-2) and V(3,-2), adapted from Mohammadi et al. (2022)

(This figure shows two images of the crack meter gridlines. The one above is at the zero and zero marks. The one below shows that the zero lines have now moved.)

Exploratory opening made in façades – Making exploratory openings in façades is a common non-destructive evaluation (NDE) technique used to assess concealed conditions within wall assemblies. The work is typically performed by a qualified contractor under the direct supervision of the professional of the record – usually the architect or engineer responsible for the façade

inspection. To create an opening, one or more façade units (such as brick, CMU, stone, terra-cotta, cladding panels, or rainscreen elements) are carefully removed to expose the backing wall, anchorage systems, and adjacent components for visual examination. The objective is to identify concealed deterioration, such as corroded anchors, failed attachment rods, or other hidden defects that may compromise structural integrity or safety. Once the inspection is complete, the removed façade units are reinstalled in their original position, and the joints are repointed or sealed to match the surrounding finish. A key consideration in this process involves determining the appropriate number and location of exploratory openings required to develop a representative understanding of the façade's concealed condition. Currently, there are no standardized or published guidelines defining the number or distribution of such openings for various façade types.

Because removing façade units – especially terra-cotta or stone elements – is costly and may require specialized handling, determining the optimum sample size is important. Too few openings may yield inconclusive or non-representative results, while too many can be unnecessarily expensive and intrusive. A phased or adaptive approach is often recommended. The investigation may begin with a limited number of openings in areas showing signs of distress or moisture infiltration. If no damage is detected, further openings may be unnecessary. However, if deterioration is observed, the percentage of affected areas can serve as a statistical basis for expanding the investigation to additional zones.

It is also essential to review applicable local façade inspection codes, as certain jurisdictions specify a minimum percentage of façade area or a required number of representative openings to be examined during an investigation. Adherence to these requirements ensures that the evaluation is both technically sound and compliant with regulatory standards. Beyond code compliance, the overall condition of the façade and the extent of observed distress play major roles in determining how many exploratory openings should be made. When a façade exhibits a large number of visible issues, yet exposing all locations is cost-prohibitive, a statistical sampling approach can be employed. In such cases, a representative subset of the total number of distress locations is selected for opening and evaluation. The findings from these samples – specifically, the percentage of damaged or deteriorated components – can then be used to guide whether additional openings are warranted.

To implement the statistical sampling method, we recommend using a binomial-type probability distribution model. Considering this model, it is possible to arrive at an equation that can be useful in determining the number of openings, especially as new information becomes available. This may especially be applicable to situations where there is an attachment rod behind each block (such as in terra-cotta façades); and the condition of these rods is being investigated. With the binomial distribution model, a parameter p is used to define the probability that a block will have a damaged connection rod; and this value is assumed to be constant for all blocks. Furthermore, it is assumed that conditions among blocks are independent. In other words, it is assumed that if one block has a damaged attachment rod, there is no reason to believe that another block will have damaged rod. Using the binomial distribution model, the following equation for the sample size can be obtained, see, for example, Khsity et al. (2012).

$$n = \frac{1-p}{p}\left(\frac{k_{\alpha/2}}{\Delta}\right)^2 \tag{8.5.1}$$

In which, n is the number of openings, p is the probability of openings with distress condition (e.g., damaged attachment rod), Δ is equal to $\pm$ variation in p, $k_{\alpha/2}$ is the value corresponding to $(1-\alpha)$ confidence level using the normal probability function – it is a value for which the probability is $(1-\alpha/2)$. Values of $k_{\alpha/2}$ for different confidence levels are provided in Table 8.5.

Table 8.5 Values of $k_{\alpha/2}$ for different confidence levels

Confidence level (1-α)	α	$(1-\alpha/2)$	$k_{\alpha/2} = \Phi^{-1}(1-\alpha/2)$
0.95	0.05	0.975	1.96
0.98	0.02	0.990	2.33
0.99	0.01	0.995	2.58

0.995	0.005	0.9975	2.81

In Table 8.5, Φ is the standard normal probability distribution function; while Φ^{-1} (phi inverse) is the inverse of this function. This means that $\Phi(k_{\alpha/2})$ is equal to $(1-\alpha/2)$ indicating that the value of the normal distribution function Φ (phi) is equal to one minus alpha divided by 2, where one minus alpha is the confidence level. The sampling error in Equation 8.5.1 is

$$\delta = \pm\sqrt{\frac{p(1-p)}{\Delta}} \tag{8.5.2}$$

Illustrative example – Example 8.1 in Appendix A illustrates the applicability of Equations 8.5.1 and 8.5.2 in making a decision on the number of openings in a façade inspection project.

In a more elaborate approach, Mohammadi and Modares (2016) introduced a statistical updating process based on the Bayesian formulation. This process uses the new findings in the number of blocks with damaged attachment rods to update p that can then be used to obtain a better estimate for n.

Water test – Water tests are conducted for investigating the sources of leaks, especially around windows. Leaks often occur in very peculiar way; and as such, they may be difficult to determine what their sources are. Generally, it is a good idea to look for leaks from the following potential sources.

- Rain water through windows and patio doors.
- Accumulated water on the roof migrating into lower floors.
- Accumulated water in balconies and porches.
- Damaged pipes.
- Broken water lines and waste water pipes at toilets, showers, and areas where washing machines are located.
- Excessive condensations.

Procedures for water tests are often based on the following documents.

- ASTM International, formerly known as the American Society for Testing and Materials (ASTM), documents ASTM C1601 and ASTM E2128, see ASTM (2017) and ASTM (2014) in the list of references for this chapter.
- The American Architectural Manufacturers Association (AAMA) documents AAMA 501.1 and 501.2, see AAMA (2017) and AAMA (2015) in the list of references in this chapter.

The AAMA is made up of manufacturers of windows, doors, skylights, curtain walls and storefronts and their suppliers. These large and small companies supply products for both residential and commercial buildings. As of January 2020, AAMA and another trade organization, Insulating Glass Manufacturers Alliance (IGMA) have been united into a new alliance called the Fenestration Glazing Industry Alliance (FGIA).

The steps in conducting a water test based on AAMA documents are as follows.

1. Cover at least 100 square feet (9.3 square meters) of area for test.
2. Include surface, window edges, window frames, sills, lintels, upper and lower portions of windows.
3. Cover about 5-foot (1.5-meter) of area at a time.
4. Water must be sprayed with a pressure of 30 to 35 pounds per square inch (0.21 to 0.24 megapascals).
5. Use a brass nozzle of one-half of an inch (13 millimeters) in diameter.
6. Spray the 5-foot (1.5-meter) area back and forth for 5 minutes, the nozzle must be taken at 1-foot (0.3-meter) from the surface.
7. Have a person present at the other side (inside) to monitor if water is leaking.
8. Any amount of water 0.5 ounces (14 grams) or more leaked inside constitutes that the test has failed.

Methods for detecting leaks from pipe and sewer line failures – In addition to the water test described, several advanced methods for leak detection can be employed, particularly in cases of pipe and sewer failures. These techniques primarily utilize sensors to detect moisture and temperature variations, or changes in response characteristics of a pipeline due to cracks and breakage. And as such, they are suitable for scenarios where water pipes are embedded in concrete, floor slabs, or otherwise inaccessible. Notably, these methods often eliminate the need for excavation, where pipes and sewer lines are located deep underground, thus reducing disruptions in traffic. Key techniques include infrared thermography, acoustic emission analysis,

ultrasonic wave detection, and ground-penetrating radar (GPR). However, these methods are typically expensive and require specialized equipment as well as trained personnel for accurate data interpretation.

Table 8.6 provides a summary of these methods, their applications, advantages and disadvantages. Additional information about these methods can be found in, for example, Islam et al. (2022) and Yussof and Ho (2022).

Table 8.6 Sensor-based methods for leak detection in pipes and sewer lines

Method	Underlying principle	Specific applications	Advant
Thermal imaging (infrared thermography)	Detecting temperature differences caused by evaporating water	Locating leaks in pipes, walls, or floors – residential and commercial buildings	Advant Disadv areas v times c
Acoustic emission	Detecting sound waves or vibrations caused by water escaping through cracks in pipes	Detecting leaks in underground water mains and pipelines	Advant pipelin Disadv leaks i
Ultrasonic sound waves	Ultrasonic sound waves are transmitted through pipe, leaks detected by analyzing changes in wave reflections	In facilities with inaccessible or embedded pipes	Advant detecti Disadv by nois
Ground penetrating radar (GPR)	Using differences in dielectric properties of materials that can indicate presence of moisture	In detecting leaks in water mains and sewer lines in urban infrastructure systems	Advant Disadv soil typ

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8.6: VIBRATION ANALYSIS

Vibration monitoring and the type of expertise needed to compile and process data were briefly discussed in the previous sections. In this section, additional details on vibration monitoring including engineering analyses that are often accompanied with the compiled data are provided. Vibration monitoring may be necessary when a steady source of vibrations is present triggering walls and wall components and roof and floor beams to the extent that they sustain damage or interfere with occupants' comfort. Examples of sources of vibrations are summarized below (also see Longinow and Mohammadi, 2022).

- Underground subway train lines.
- Construction equipment working for an extended period of time.
- Aircraft take-offs.
- Busy highways, feeder ramps, bridges with heavy truck and train traffic.
- Heavy mechanical systems and machines continuously operating in the building.
- Wind effects with frequent wind speed fluctuations.
- Earthquakes.

The effect of vibrations is a continuous fluctuation of deformation, which results in reversal of stress function, especially if there is a large initial stress (e.g., dead load stress) present. This results in cracks. As indicated earlier, accelerometers are used to measure vibrations. However, care must be exercised to select an accelerometer with adequate sensitivity to measure low acceleration levels and very low frequency, which are characteristics of vibrations in façades and other structural systems. Figure 8.4 illustrates an example of stress fluctuations that occur as a result of vibrations. Although the initial stress is in compression (considering compression as positive), as a result of fluctuations, the stress will have cycles of tension. Thus, triggering a masonry wall, these frequent tension stress cycles may initiate and promote cracks.

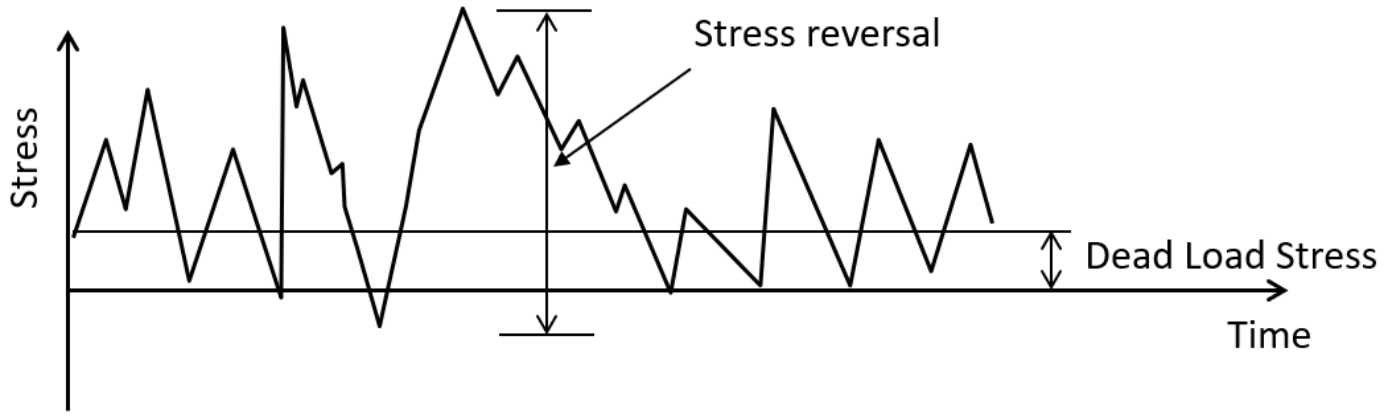


Figure 8.4 Fluctuation of stress because of vibrations

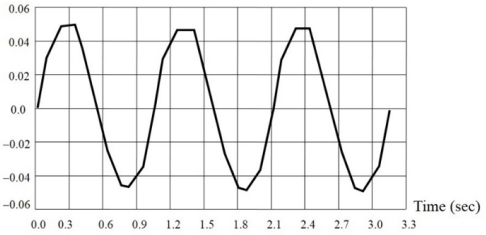
(This figure shows an example of how stress fluctuates in time.)

In the case of structural elements such as lintels, floor joists, and roof joists, the extent of damage resulting from dynamic loading cannot be accurately assessed without conducting a detailed structural dynamic analysis. This analysis typically requires using a recorded vibration signal (e.g., as shown in Figure 8.4) as the input excitation to the structural model. However, a preliminary assessment can be made by calculating the fundamental natural frequency of the beam and ensuring that this frequency does not coincide with the dominant frequency of the applied excitation. If the beam's fundamental natural frequency (often referred to as the resonance frequency) matches or closely approximates the frequency of the external input, resonance may occur. Resonance significantly amplifies the vibration response, which can lead to accelerated structural degradation, damage to adjacent components (such as walls), or discomfort for building occupants.

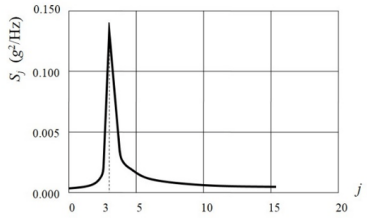
To analyze the characteristics of an input vibration, the time history record – such as the one illustrated in Figure 8.4 – must be transformed from the time domain into the frequency domain. This is achieved through a mathematical operation known as the Fourier Transform. In practical applications, because vibration time histories (e.g., acceleration data) are typically available in discrete form, this transformation is performed numerically using computational tools. The numerical implementation is

commonly referred to as the Fast Fourier Transform (FFT). Applying an FFT to time-domain data yields a frequency-domain representation, where the resulting values correspond to the ordinates of a function known as the Power Spectral Density (PSD). The PSD describes how the power of a signal is distributed across different frequency components.

As an illustration, Figure 8.5 presents two related plots: Figure 8.5(a) shows a sample acceleration time history, which might be generated by a machine operating at a constant amplitude – such as a building's HVAC system in steady-state operation. Figure 8.5(b) displays the corresponding PSD function S_j of the same record. As seen in this figure, there is only one frequency in the record, which is equal to j = 3. As explained later, the parameter j is not a real frequency but is linearly related to it. In fact, if we divide j by the total period of the record used in FFT, we get the cycles-per-second frequency (which is in hertz). As seen in Figure 8.5(a), the time-history record is 3.1 seconds long, therefore, the real frequency f = 3 divided by 3.1 seconds, which results in 0.97 hertz. However, it is noted that in some software output the parameter j (or like) is mentioned also as frequency.



(a) Example of an acceleration time history of vibration from a machine with constant amplitude

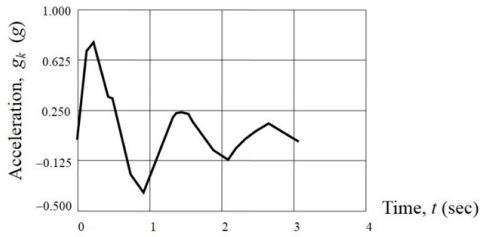


(b) Power Spectral Density (PSD) Function for time history in (a), dominant frequency is at *j* around 3

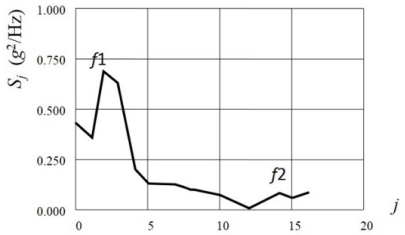
Figure 8.5 Examples of a constant-amplitude vibration and the corresponding PSD, adopted from Khisty et al. (2012)

(This figure shows an image of a constant amplitude time history at the top. Below this, an image of the power spectral function is shown with one spike at around *j* equal to 3, which related to the dominant frequency.)

Of course, the appearance of the single frequency in the record of Figure 8.5 is not surprising because the record has a constant amplitude with a constant period of vibration. In a more complex record, the dominant frequency will not be easily detectable unless the PSD graph is formed. As an example, Figure 8.6 presents an acceleration record with mixed frequencies. The time history shown in Figure 8.6 (a) does not show a dominant frequency. However, upon using a FFT and obtaining the PSD, as shown in Figure 8.6 (b), at least two frequencies ($f1$ and $f2$) are dominant in the record.



(a) Time history record of acceleration with mixed frequencies



(b) PSD of acceleration, notice at least two frequencies appear, *f*1 (at *j* =2) and *f*2 (at *j*=14)

Figure 8.6 Example of an acceleration record with mixed frequencies, adopted from Khisty et al. (2012)

(The image at the top is the time history of a sudden jolt to a shaking table lasting 3 seconds. The image below shows the equivalent frequency diagram with two spikes at $f1$ and $f2$ frequencies.)

Relevant equations in FFT – In a mathematical form, the Fourier Transformation equation pair are written as follows (notice that the PSD is in terms of the angular frequency ω and recall that the frequency f in cycles per second (hertz) is equal to $\omega/(2\pi)$ (in other words, omega divided by two pi).

$$S(\omega) = \frac{1}{2\pi}\int_0^\infty g(t)e^{-i\omega t}dt \tag{8.6.1}$$

$$g(t) = \int_0^\infty S(\omega)e^{i\omega t}d\omega \tag{8.6.2}$$

In these equations, $S(\omega)$ isthe PSD, $g(t)$ is the time history of acceleration, t is time, and i is equal to the square root of negative 1 ($i = \sqrt{-1}$). Since usually we have a series of data points in discrete forms, a numerical equation is used to find the values of S, in the following form (Khisty et al., 2012):

$$S_j = \frac{1}{\sqrt{n}}\sum_k g_k exp\left[i\left(2\pi\frac{j}{n}\right)k\right] \tag{8.6.3}$$

In which, n is the number of sample points for the time history function, g_k is the kth value of the time history function, S_j is the value of the PSD function at j^{th} frequency. In this equation, all g_k values are compiled at equal time intervals. Furthermore, the number of sample points n has to be in the form $n = 2^m$ and the number of S_j points from the transformation (Equation 8.6.3) will be 2^{m-1}. For example, if n is equal to 128 (which is 2 to the power of 7), the S_j results will have 64 sample points (which is 2 to the power of 6). Also notice that instead of the angular frequency ω, parameter j is used. As we indicated earlier, j is related to the frequency f, where

$$f = \frac{\omega}{2\pi} \tag{8.6.4}$$

Since Equation 8.6.3 does not give values in terms of the actual frequencies of the original time-dependent data set, to obtain the actual frequencies, the following equation is used.

$$f_j = \frac{j}{n}f_s \tag{8.6.5}$$

in which, f_j is the jth frequency and f_s is the sampling rate of the compiled data g_k. For example, if n is 64, and there are five samples per second compiled to obtain g_k, then f_s is 5. This means the total time period used in compiling the data $t = n/f_s$ or 64 divided by 5, which equals 12.8 seconds. Therefore, we can simply divide j by the total length of time in the time-history data set

and obtain f. Also notice that n is equal to 64 or 2 to the powers of 6. Thus, the total number of samples computed for S_j will be 2 to the power of 5 or 32. If the S_j results, for example, show a peak at j equal to 3, it means a dominant frequency of the compiled data is equal to 3 divided by 64 and multiplied by 5, which is 0.23 hertz. It is important to note that, according to this relationship, frequencies higher than the sampling frequency f_s cannot be detected or accurately resolved. Therefore, to capture the full range of frequency content present in the signal, the sampling frequency must be sufficiently high. This ensures compliance with the Nyquist Criterion (see for example Shannon, 2005), which states that the sampling frequency should be at least twice the highest frequency present in the signal, to avoid aliasing and to provide a meaningful frequency-domain representation.

As it is evident from the FFT equations, values obtained for S_j are complex numbers. Thus

$$S_j = Re(S_j) + iIm(S_j) \tag{8.6.6}$$

Where Re is the real and Im is the imaginary part of the complex number S_j. To plot the results, we usually use the following equation

$$|S_j| = \sqrt{[Re(S_j)]^2 + [Im(S_j)]^2} \tag{8.6.7}$$

In practice, the PSD function is automatically generated using the built-in processors in the data acquisition system. If the raw data on acceleration is compiled and the FFT needs to be applied, the easiest method is to use either MathLab®, MathCad®, or the Microsoft Excel®. While using the data in Microsoft Excel, notice that if there are 2 to the power of m sample data, the PSD function will have 2 to the power of m minus one. However, the software will still give 2 to the power of m samples. It is important to note that only 1/2 of the data in the PSD column is used.

Illustrative example – Example 8.2 in Appendix A provides an illustration of how the dominant frequency in a vibration record is obtained using the FFT method.

8.6.1: Computing fundamental natural frequency of vibration of beams

8.6.2: Retrofitting a beam or lintel to control vibration

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8.6.1: Computing fundamental natural frequency of vibration of beams

Beam fundamental frequency – As discussed earlier, one of the most critical effects of vibration on beams occurs when the excitation (input) frequency approaches the beam's fundamental natural frequency. Under this resonance condition, even small dynamic loads can induce large amplitude vibrations, leading to excessive deflections, crack formation, or even structural failure. In practical applications, when resonance is suspected or identified, the structural design must be modified to alter the beam's natural frequency and eliminate the potential for resonance. And as explained earlier, this is typically achieved by increasing the beam's stiffness – for example, by selecting a stronger, or reducing the span length by introducing additional supports – to raise its natural frequency beyond the excitation range. This section presents simplified analytical methods for estimating the fundamental natural frequency of beams (such as and lintels), which can be used for preliminary evaluation and design adjustments prior to detailed dynamic analyses.

The fundamental natural frequency of a beam represents the number of oscillation cycles the beam undergoes per unit time (typically expressed in hertz, or cycles per second) when it is displaced from its equilibrium position and allowed to vibrate freely. Due to the inherent material property known as damping, these vibrations gradually decay until motion ceases. In theory, a beam possesses an infinite number of natural frequencies, each corresponding to a specific mode shape of vibration. The overall dynamic response of the beam is a superposition of these mode shapes. However, the fundamental (lowest) natural frequency is generally the most critical, as it governs the dominant response of the structure under dynamic excitation and is most susceptible to resonance effects. While detailed structural dynamic analysis – often using finite element methods – is required for precise determination of natural frequencies, simplified analytical expressions can provide reasonably accurate estimates for preliminary evaluation. For instance, in the case of a simply supported beam (such as a lintel), the fundamental mode corresponds to the deflected shape forming a single half-wave curvature between the supports.

For a beam undergoing oscillations, an approximate equation for the fundamental natural frequency f of vibration is

$$f = \frac{1}{2\pi}\sqrt{\frac{K}{M}} \tag{8.6.1.1}$$

In which, K is the stiffness and M is the mass. The stiffness can simply be obtained as the ratio of an applied load P to the beam deflection Y.

$$K = \frac{P}{Y} \tag{8.6.1.2}$$

For simply-supported and cantilever beams under various loading situations, the corresponding deflections Y and the stiffness values are given in Table 8.7. Recall that the mass M is the weight W divided by the gravity acceleration g, where the gravity acceleration g is 386 inches per second squared (9.8 meters per second squared). The total load is made up of the dead load only (without using the load factors).

In Table 8.7, L is the span length, I_x is the moment of inertia of the beam cross section, q is the beam linear load (in pounds per foot or kilonewtons per meter); while P is the total dead load (in pounds or kilonewtons). The unit for K is generally in pounds per inches or kilonewtons per meters.

Table 8.7 Equations of deflection Y and stiffness K for simply-supported and cantilever beams of span length L and moment of ine

Beam condition and loading	Deflection Y	Stiffness K
Simply-supported beam with concentrated load	$\frac{PL^3}{48EI_x}$	$\frac{48EI_x}{L^3}$
Simply-supported beam with uniform load q and total load $P = qL$	$\frac{5PL^3}{384EI_x}$	$\frac{348EI_x}{5L^3}$

Simply-supported beam with an isosceles triangular load of a peak q and total load $P = qL/2$	$\frac{PL^3}{120EI_x}$	$\frac{120EI_x}{L^3}$
Cantilever beam with a concentrated load P	$\frac{PL^3}{3EI_x}$	$\frac{3EI_x}{L^3}$
Cantilever beam with a uniform load q and total load $P = qL$	$\frac{PL^3}{8EI_x}$	$\frac{8EI_x}{L^3}$

Illustrative example – Example 8.3 in Appendix A illustrates the simple process of computing frequency of vibration of a beam.

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8.6.2: Retrofitting a beam or lintel to control vibration

In situations where the vibration because of resonance will need to be controlled, the solution is to increase the fundamental frequency of the beam so that it will be out of the phase with the frequency of the input vibration. This can mainly be achieved by using a stronger beam with a larger cross-sectional moment of inertia to increase f. Another option would be to use an identical beam attached to the existing one. This will double the moment of inertia. Notice that by doubling the moment of inertia, the stiffness is also doubled, however, the frequency will be increased by a factor of $\sqrt{2}$ = 1.4. Depending on the situation, we may decide to reduce the span length by changing the design of the beam. Reduced span length will increase the stiffness substantially. Another solution is to either eliminate or isolate the source of vibration. For example, an air conditioning unit that causes vibrations in a lintel can be moved to another location, where the supporting floor or roof is stiffer, or the unit can be mounted on vibration isolating devices.

For additional reading regarding the vibration effects, AASHTO (2009), ANSI (2012), and papers by Jaafari and Mohammadi (2018) and Longinow and Mohammadi (2022) are recommended. In their paper, Jaafari and Mohammadi (2018) discuss vibrations in floors within the context of the serviceability control. They further provide and overview of methods used for vibration isolations. The paper by Longinow and Mohammadi (2022) provides and overview of vibrations in building and the common sources of vibrations. Furthermore, several case studies are presented and the types of solutions that can be used to abate the damaging effects of vibrations are explained.

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8.8: Exercises

E8.1 The acceleration data (in g unit) at a façade location for 6.3 seconds is provided in the table below. Notice that n is 64 and sampling rate f_s is equal to 10 data points per second.

a) Plot the time history for this record.

b) Conduct a Fast Fourier Transformation (FFT) on this data, and plot the results.

c) Estimate the critical frequency of the record.

Acceleration data

t (second)	$X(t)$	t (second)	$X(t)$
0	0	3.2	-0.0041
0.1	0.00103	3.3	-0.00873
0.2	0.04722	3.4	0.013168
0.3	0.037008	3.5	0.02914
0.4	0.006393	3.6	0.046564
0.5	0.057081	3.7	0.029798
0.6	0.005154	3.8	0.034675
0.7	0.032395	3.9	0.032028
0.8	0.00667	4	0.002762
0.9	-0.00771	4.1	0.014914
1	-0.03404	4.2	-0.0049
1.1	-0.03044	4.3	-0.00728

1.2	-0.04544	4.4	-0.03241
1.3	-0.07715	4.5	-0.02311
1.4	-0.06335	4.6	-0.00211
1.5	-0.0172	4.7	-0.05581
1.6	-0.01244	4.8	-0.02125
1.7	0.003048	4.9	-0.01353
1.8	0.019711	5	-0.00051
1.9	0.016648	5.1	0.005373
2	0.014845	5.2	0.01219
2.1	0.037445	5.3	0.039104
2.2	0.045248	5.4	0.011464
2.3	0.019108	5.5	0.003628
2.4	0.015451	5.6	0.008435
2.5	4.79E-05	5.7	0.019276
2.6	-0.01961	5.8	0.007173
2.7	-0.03354	5.9	-0.00791
2.8	-0.03918	6	-0.00851

2.9	-0.02369	6.1	-0.02921
3	-0.00999	6.2	-0.03352
3.1	-0.00624	6.3	-0.03387

E8.2 Estimate the natural frequency of vibration of a steel beam used in a balcony as a cantilever system. The span length is 8 feet (2.4 meters), the section is a channel with a cross-sectional moment of inertia of 52.4 inches to the fourth power (21.8 times 10 to the power of 6 using millimeters to the fourth power as the unit). The dead weight carried by this beam q is 120 pounds per foot (1.8 kilonewtons per meter).

E8.3 What is the most probable reason the brick wall above a lintel may crack when the lintel vibrates a lot?

a) The brick has poor mortar quality.

b)The vibration may cause fluctuations of stress in masonry into tension cycles.

c) The lintel's end supports are too wide.

d) The cross section of the section used as the lintel has a lower yield capacity than usual.

E8.4 For a lintel with a steel cross section that vibrates because of an air-conditioning unit running, a solution is to increase the natural frequency of vibration. Which of the following changes in design is most effective in increasing this frequency?

a) Doubling the cross-section moment of inertia while keeping the span length the same.

b) Reducing the span length to one half of the original while keeping the cross-section the same.

c) Changing the lintel with an identical beam of the same length and cross section but with a larger yield resistance.

d) Reducing the dead load by 50% and keeping everything else the same.

E8.5 A façade is made up of terra-cotta blocks that are suspicious of having corroded attachment rods. From experience, we know that in other situations, walls similar to this have blocks with 35% probability that the attachment rods behind them are corroded. The coefficient of variation in estimating this probability is ±0.28.

a) At 0.98 confidence, and using a statistical sampling method, determine an estimate for the number of blocks that will need to be removed for inspection.

b) Compute the sampling error (using percentages) in regard to the estimate in part (a).

E8.6 An NDT method called core sampling is generally used in

a) Identifying the moisture contents of a material.

b) Identifying the porosity of a material.

c) Finding concrete mechanical properties and conducting petrographic studies.

d) Finding the thermal properties of a material.

E8.7 In the table below, four NDT methods are listed in Column 1 and labeled as Methods A, B, C and D. In Column 2, four applications listed labeled as Applications 1, 2, 3 and 4. Match each Method to its most appropriate application.

NDT methods and Applications

Column 1: NDT method	Column 2: Applications
Method A: Ground penetration radar (GPR)	Application 1: Detection of leaks through wall
Method B: ASTM E2128	Application 2: Wall movement measurement
Method C: Air sampling test	Application 3: Locating rebars in concrete
Method D: Camera installed afar aiming at targets on building	Application 4: Mold detection

E8.8 Which of the following statements is least accurate?

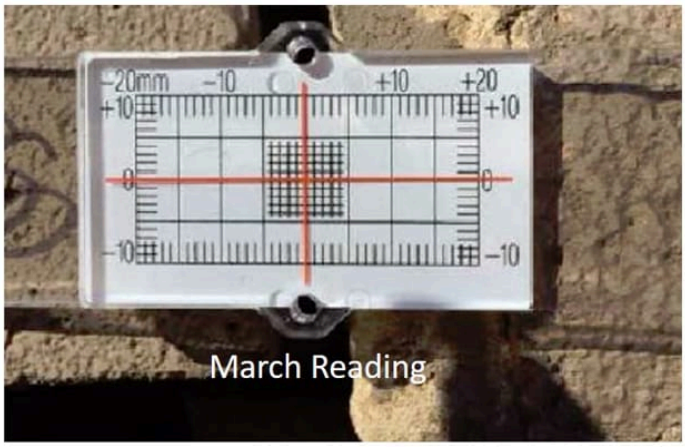
a) All professionals involved in asbestos handling projects need to be licensed in their respective expertise.

b) Asbestos is a highly hazardous material and requires experts handling them.

c) A licensed expert who identifies, assesses the condition of, or collects bulk samples of suspected asbestos-containing materials is called an asbestos inspector.

d) Architects or engineers in charge of façade inspection and rehabilitation projects can mostly handle asbestos situations by themselves as long they hold a professional engineering or architectural registration.

E8.9 Images of a crack meter at two different times are provided in the accompanying figure. Provide your readings for these two images using H(x1,x2) and V(y1,y2) format. Comment whether there have been any changes in the crack condition. Note, H means horizontal movement of the vertical red line; while V means vertical movement of the horizontal red line.





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9: ECONOMICS OF BUILDING ENVELOPE REPAIR, PLANNING STRATEGIES, RETURN ON INVESTMENT, RESERVE STUDY

Chapter Synopsis

This chapter explores how engineering economic analysis supports sound decision-making in building envelope inspections and evaluations. Readers are introduced to methods for comparing alternatives over short-, medium-, and long-term planning horizons, with attention to balancing costs and benefits, including return on investment. The discussion also highlights reserve studies, a key tool used to plan for the timely repair, rehabilitation, and replacement of shared components in condominiums and other multi-unit housing complexes.

9.1: Introductory Remarks

9.2: Importance of Economic Analysis in Façade Repair, Rehabilitation, and Replacement

9.3: Possible Options for Façade Repair, Rehabilitation, and Replacement

9.4: Basic Equations of Engineering Economics for Evaluation of Alternative and Reserve Study

9.4.1: Nomenclatures and equations

9.4.2: Evaluation methods

9.5: Reserve Study

9.5.1: Objectives of a reserve study

9.5.2: Phases of a reserve study

9.6: Chapter References

9.7: Exercises

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9.1: Introductory Remarks

This chapter presents an overview of engineering economic analysis as it applies to building envelope repair and rehabilitation. In general, such evaluations are not part of routine inspection or repair projects unless specifically required by contract. However, two scenarios commonly necessitate their use. The first involves the evaluation of alternative repair strategies, particularly when comparing the costs and benefits of immediate intervention versus deferring repairs to a later date. The second arises in the context of reserve studies, where financial assessments are performed to ensure that adequate funds are allocated for the timely repair, rehabilitation, and replacement of common area components in condominiums and other residential complexes.

Evaluation of alternatives – Owners and property managers are often interested in exploring alternatives to the specific repair or rehabilitation strategy recommended for their properties. Except in cases where urgent repairs are required due to immediate safety concerns, there are usually multiple viable options available for building envelope repair and rehabilitation. In many projects, financial considerations play a central role in determining the timing and scope of work. For example, in apartment or condominium complexes, approval from unit owners may be required to raise funds through special assessments. This process often introduces delays, as residents must reach consensus and secure the necessary resources. In such situations, it is critical that owners understand what alternatives exist, beyond immediate repair, and the implications of each option.

Engineering economic analyses provide a systematic framework for comparing the costs, benefits, and long-term implications of various alternatives. Importantly, some owners may not fully appreciate the urgency of repairs unless prompted by a citation from the municipal building department for safety violations. In such cases, the owner is obligated to retain a qualified professional to assess the condition of the building envelope and provide appropriate recommendations. To support sound decision-making, the professional's responsibility extends beyond documenting technical conditions: they must also present the projected costs, potential savings, and relative benefits associated with different courses of action.

One of the most common alternatives owners inquire about is the financial impact of deferring repairs when no critical safety issues are present. This option, often referred to as the "do-nothing" alternative, must be carefully evaluated and its long-term economic consequences clearly communicated. Engineering economic analyses are particularly useful here, as it allows professionals to quantify how deferred actions may lead to accelerated deterioration, increased future repair costs, or even reduced property value. In many cases, analyses demonstrate that borrowing funds to complete repairs promptly may be more cost-effective than delaying the work.

By systematically evaluating alternatives - including immediate repair, phased implementation, borrowing, or the do-nothing option – owners gain the flexibility to align repair strategies with their financial capacity, while still maintaining building performance and safety. When communicated effectively, this type of analysis equips owners with the knowledge required to make informed, economically sound decisions regarding their building envelope.

Reserve study – A reserve study applies engineering economic principles to anticipate and plan for the future repair and replacement of common area components in condominiums and other multi-housing complexes. While such studies address a wide range of building systems and equipment, a substantial portion of the analysis typically pertains to façades and their associated components. Reserve studies are widely recognized as essential financial planning tools, enabling property owners, homeowners' associations, and condominium associations to ensure that adequate funds will be available for major repair and replacement projects. In fact, several states mandate reserve studies for residential buildings of three stories or greater, with updates generally required every 3 to 5 years.

A comprehensive reserve study involves collecting detailed information on the expected service life and replacement cost of all shared building systems and amenities. These components may include mechanical equipment, recreational facilities, swimming pools, landscaping, roofs, façades, and other exterior elements. Once this information is compiled, engineering economic methods – such as life-cycle cost analysis and present worth calculations – are applied to project the timing and magnitude of required expenditures. This ensures that adequate financial reserves are established in advance, thereby preventing unexpected costs from creating a financial burden on the property owners or association.

In the context of this book, emphasis is placed on reserve study applications related to building envelope systems. The chapter provides an overview of fundamental engineering economics equations used in evaluating alternatives, with particular focus on how these tools support decision-making, and capital reserve accumulation, for timely repair, rehabilitation, and replacement of façades, roofs, and related components.

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9.2: Importance of Economic Analysis in Façade Repair, Rehabilitation, and Replacement

Although not typically required, including an engineering economic study alongside an inspection report can provide critical information when an owner must determine how to finance and plan recommended repairs, rehabilitation, or replacement. Such a study, particularly in the context of façade repair and rehabilitation, focuses on comparing alternative approaches to the work. It may be incorporated into the broader façade evaluation process at an early stage of the inspection or conducted later as part of a "due diligence" exercise. In building envelope investigations, due diligence refers to the systematic inspection, evaluation, and analysis of the envelope's condition prior to preparing a report for the owner or other stakeholders.

The value of an engineering economic study lies in its ability to demonstrate the importance of maintaining the safety and integrity of the building envelope while providing owners with clear cost projections for both short- mid- and long-term scenarios. Because such studies are not generally included as part of the inspection process, it is essential to discuss their potential benefits with the owner at the outset. If agreed, the scope should be explicitly included in the contract. Conducting the analysis requires project-specific data, evaluation of multiple alternatives, and preparation of a separate report. Early discussions ensure that the additional effort is properly estimated, budgeted, and scheduled.

The inputs and background knowledge required for an engineering economic study include:

- Understanding of repair needs and associated costs - including direct labor and materials as well as indirect costs. Indirect costs may involve penalties for delayed repairs or the expense of temporary safety installations during construction.
- Consideration of short-term versus long-term repair needs. Short-term work (3–12 months) typically addresses critical safety concerns, which may also necessitate temporary measures to protect occupants and the public. Long-term work is generally scheduled beyond the inspection period and is documented in the inspection report.
- Familiarity with engineering economic analysis methods to evaluate costs and benefits of different options.
- Information on inflation rates, interest rates, and the expected service lives of materials and equipment.
- Awareness of requirements by various states and municipalities for reserve studies and applicable standards of practice, such as those recommended by CAI (2023).

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9.3: Possible Options for Façade Repair, Rehabilitation, and Replacement

If there is no immediate safety concern, typical options (alternatives) may include the following:

Option 1: Doing-nothing – This alternative consists of deferring major repair or rehabilitation work to a later date. However, a systematic program of inspection and preventive maintenance must still be implemented to ensure the continued performance of the building envelope. Such a program typically includes routine activities such as annual re-caulking and sealant replacement, repainting of exposed surfaces, cleaning of drainage components, and removal of accumulated debris. For large or complex facilities, it is often practical to divide the building envelope into manageable zones or segments, with inspection and maintenance operations scheduled on a rotational, multi-year cycle. This approach enables continuous upkeep while distributing costs and labor over time. Although this strategy may initially appear to be the more economical option, the owner should be clearly informed that significant capital repair or replacement will eventually be required. In essence, this approach allows the owner to "defer investment" or "buy time", but economic analysis often demonstrates that postponement may result in higher overall lifecycle costs due to progressive deterioration and inflationary factors.

Option 2: Dividing the required work in phases – Under this option, the required repair and rehabilitation work is implemented in multiple phases over a defined time period. A detailed phased work plan is developed to outline the scope, schedule, and nature of activities to be performed during each phase. For instance, one section of the façade may be designated for repair in Year 1, another in Year 2, and so forth, following a structured multi-year (e.g., five-year) plan. With this approach, the financial expenditure is distributed across several years, allowing the owner to manage costs more predictably. However, the total cumulative cost typically increases due to factors such as inflation, material escalation, and the continued deterioration of unrepaired areas. Moreover, portions of the façade awaiting future repair will still require periodic maintenance, stabilization measures, and localized repairs to ensure safety and prevent accelerated damage. This phased strategy is particularly suitable for large multi-tenant or institutional buildings, where work can be coordinated to minimize operational disruptions. Funding for each phase may be secured through capital improvement budgets, special assessments, or incremental rent adjustments, allowing the owner to accumulate the necessary resources as successive phases are executed.

Option 3: Doing the entire required repair now – This option involves executing the entire scope of required rehabilitation work as a single, comprehensive project, funded through a major upfront capital investment. The owner must secure the necessary funds – either from existing capital reserves or through financing mechanisms such as loans or bonds – to cover the total project cost. Although project mobilization and implementation may take several months to a year, for purposes of economic and life-cycle cost analysis, the total expenditure is considered a present worth value (as discussed later). Upon completion of the project, major rehabilitation expenses are effectively eliminated for the foreseeable future, with only routine maintenance activities – such as cleaning, sealant replacement, or minor repairs – remaining as recurring costs. When evaluating this option, it is also important to consider potential returns on investment (ROI) resulting from the work. For example, replacement of deteriorated or inefficient façade components, such as windows or glazing systems, can lead to measurable energy savings, improved thermal performance, and enhanced occupant comfort. Additionally, the upgrade may yield secondary financial benefits, including potential reductions in insurance premiums or eligibility for sustainability incentives tied to energy efficiency improvements.

Other options including hybrid strategies for repair and rehabilitation – As expected, hybrid strategies may also be developed that combine aspects of the aforementioned three primary repair and rehabilitation options. For instance, an owner may elect to undertake a portion of the critical repairs immediately – addressing the most urgent deficiencies – while deferring less critical work to a later phase. This blended approach can help balance technical priorities, budgetary constraints, and operational considerations. Regardless of the selected strategy, it is essential that the owner fully understands the implications of each alternative. The following key points should be clearly communicated during the planning and decision-making process.

- All necessary work will ultimately need to be completed. Deferring essential repairs may temporarily reduce immediate costs, but deterioration will continue, potentially increasing the scope and cost of future work.
- The optimal option is the one that best aligns with the owner's financial capacity, risk tolerance, and long-term facility management goals. What may appear to be a cost-saving decision in the short term could lead to accelerated degradation and higher cumulative expenditures.
- Apparent short-term savings can mask higher long-term costs. Deferred maintenance and partial repairs can result in compounding deterioration, more complex future interventions, and potential liability or safety risks.

Ultimately, a well-informed decision should be based on a comprehensive life-cycle cost analysis that accounts for both immediate expenditures and long-term performance, ensuring that the chosen strategy provides the best value over the service life of the building façade system.

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9.4: BASIC EQUATIONS OF ENGINEERING ECONOMICS FOR EVALUATION OF ALTERNATIVE AND RESERVE STUDY

and involves evaluating multiple alternatives – ranging from immediate to deferred action or, in limited cases, taking no corrective measures, as explained earlier. The selection among these alternatives must always be grounded in engineering judgment, risk assessment, and economic considerations. A primary and non-negotiable factor in evaluating all alternatives is safety. Ensuring public and structural safety is both a regulatory obligation and a critical cost component. It cannot be compromised or deferred, as doing so may result in severe consequences, including potential failures or liability exposure. Therefore, any decision to postpone a recommended should be limited strictly to situations where deferral can be justified without compromising safety or accelerating deterioration. In cases where safety concerns have been adequately addressed, and the owner seeks guidance on selecting an approach that best aligns with available financial resources, principles of engineering economics become particularly useful. These methods provide a quantitative framework for comparing and alternatives, enabling rational and transparent decision-making based on life-cycle cost analysis and present worth evaluation. This section presents the fundamental equations and concepts of engineering economics as applied to and planning, including guidance on reserve studies and long-term capital planning for building management. , in addition to the above information, the replacement costs of common area components are also needed. For example, costs associated with windows and doors replacements, roof replacement, etc. Note that in doing a , often an inflation rate is mentioned. This rate appears the same as the parameter$i$ when computing the needed future funds for or replacement. However, if the includes a return in an investment, the value of the interest rate will be different from the inflation rate. For example, an association may decide to start saving their accumulated funds in a saving account that pays 4% annual return, while the inflation rate, say, is 3% per year. Therefore, the parameter i for computing the future cost of a component replacement, after the service life is exhausted, will be 0.03; while to compute the future worth of the funds in the saving account, the parameter i will be 0.04.

9.4.1: Nomenclatures and equations

9.4.2: Evaluation methods

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9.4.1: Nomenclatures and equations

In this book, the following nomenclatures are used for the cost (or return) items

A: The end-of-period cost, i.e., payment (or return). Since the unit of time used is mainly in years, this will be a yearly cost (or return).

P: Present worth, for example, the initial cost of repair.

F: A future worth, cost or return, for example, future cost of replacing the roof.

G: The gradient payment. When the end-of-period payment is increased steadily, the arithmetic gradient is used to define the change. Using G, and considering that in the first year the payment is A, with the increase, the second-year payment is $A+G$, the third-year $A+2G$ and so on, see, for example, Khisty et al. (2012).

The following basic equations convert one cost to another.

Relations between future worth F and present worth P are

$$F = P(1+i)^n \tag{9.4.1.1}$$

$$P = \frac{F}{(1+i)^n} \tag{9.4.1.2}$$

The ratio F/P is called the *single payment compound amount factor*; while P/F is called the *single payment present worth factor*.

The questions relating the end-of-period cost A with P and F are

$$A = P\frac{i(1+i)^n}{(1+i)^n-1} \tag{9.4.1.3}$$

$$P = A\frac{(1+i)^n-1}{i(1+i)^n} \tag{9.4.1.4}$$

$$F = A\frac{(1+i)^n-1}{i} \tag{9.4.1.5}$$

The terms used in describing the ratios relating A, P and F to one another are as follows.

A/F: *uniform series sinking fund factor*

A/P:*capital recovery factor*

F/A: *uniform series compound amount factor*

P/A: *uniform series present worth factor*

For the gradient series, an equivalent end-of-period cost A_{Eq} for the gradient series value G is computed using the following equation.

$$A_{Eq} = \frac{G}{i} - \frac{nG}{i}\left[\frac{i}{(1+i)^n-1}\right] \tag{9.4.1.6}$$

$$A_{Total} = A + A_{Eq} \tag{9.4.1.7}$$

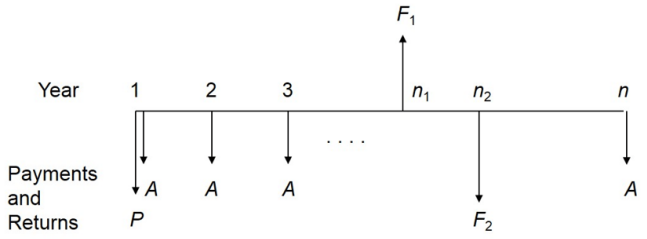
These factors are summarized in Table 9.1.

Table 9.1 Factors relating A, P, F, and G to one another (Khisty et al., 2012)

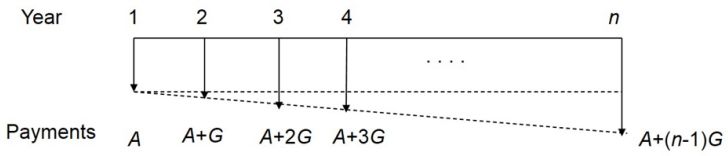
Factor	Ratio	Equation
Single payment: 1. Compound amount 2. Present worth	F/P P/F	$F = P(1+i)^n$ $P = F/(1+i)^n$
Uniform series: 3. Compound amount 4. Sinking fund 5. Present worth 6. Capital recovery	F/A A/F P/A A/P	$F = A[(1+i)^n - 1]/i$ $A = Fi/[(1+i)^n - 1]$ $P = A[(1+i)^n - 1]/[i(1+i)^n]$ $A = Pi(1+i)^n/[(1+i)^n - 1]$
Arithmetic gradient: 7. Uniform series equivalent	A/G	$A_{Eq} = \frac{G}{i} - \frac{nG}{i}[\frac{i}{(1+i)^n - 1}]$

It is further noted that, in using these equations, any cost item, which means funds to be spent, will get a negative sign; while any return, which means funds to receive because of say, an energy saving, a rebate, etc., will get a positive sign.

The flow of activities, in terms of various costs or returns, over the entire period of project can also be shown graphically using timelines. Figure 9.1 shows two examples of timeline graphs. In this figure, (a) a uniform payment series with an initial and a future expense and a future return; and (b) a gradient series are shown, with costs as an arrow downward (i.e., negative value) and a return as an arrow upward (i.e., positive value).



(a) Uniform payment (A) with present expense P, a future return F_1 and a future expense F_2



(b) Uniform payment (A) with gradient increase (G)

Figure 9.1 Examples of timeline presentation of payments, expenses, and returns

(This figure shows a time series with arrows at one-year intervals drawn downward indicating costs over time.)

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9.4.2: Evaluation methods

Four methods are used for evaluations as explained in Khisty et al. (2012). These are:

- The net present worth (NPW) method.
- Equivalent uniform annual cost (EUAC) Method.
- The rate of return method and incremental rate of return method.
- The benefit-cost analysis and incremental benefit cost analysis method.

The popular methods are the NPW and EUAC, especially in situations when do-nothing is considered as an alternative. Note that methods that use benefit cost ratios may not work for options when there is no initial cost, since a zero appears in the denominator in the benefit cost ratio.

In this chapter, the procedures and examples using the NPW and EUAC are presented. For details of the other methods, the user is referred to Khisty et al. (2012).

The NPW method – In using this method, all costs and returns for various alternatives are compiled. It is important to identify which cost (or return) item and defined them appropriately as A, P, F and G. Generally, if a cost (or return) is uniform and constantly appearing at each period, it is A; if a cost is initial (i.e., occurring now), it is P; if the cost (or return) occurs at some time in the future, it is F; and if there is a steady rate of increase in the end-of-period payment, the incremental increase is G (see Figure 9.1). The steps used in this method are as follows.

1. Convert all future worth values F and all yearly costs or benefits values A into present worth (i.e., P) values.
2. If gradient series G is involved, convert it to equivalent A first, then to present worth P.
3. Use negative signs for all costs and positive for all returns or benefits.
4. Find the algebraic sum of all the present worth values. This is called the net present worth (NPW).
5. Select the alternative with the largest NPW. Note that NPW may have a negative sign. Thus, use the NPW that is largest with consideration for any negative values it may have.

For example, in Figure 9.1(a), using the NPW methods, the individual terms

PW1 = $-P$

PW2 = $-A[(1+i)^n - 1]/[i(1+i)^n]$ this is for converting A to present worth

PW3 = $+F_1/[(1+i)^{n_1}]$ this is for converting F_1 to present worth

PW4 = $-F_2/[(1+i)^{n_2}]$ this is for converting F_2 to present worth

Total NPW = PW1 + PW2 + PW3 + PW4

The EUAC method – Again, in this method the information on all costs (and estimate of all returns, if any) with various alternatives is complied. All cost and return items are converted into equivalent end-of-period payment. The following steps describe this method.

1. Convert all present worth and future values to an equivalent uniform annul cost, i.e., convert all P, and F values to A values.
2. If a gradient G is present, find the equivalent A for it.
3. Use negative for all costs and positive for all benefits.
4. The EUAC is the algebraic sum of all *A* values.
5. Select the alternative with the largest EUAC. Again, the final EUAC for the alternative may be negative, so select the largest value considering the sign of EUAC.

For example, in Figure 9.1 (b), the EUAC values are

EUAC1 = $-A$

EUAC2 = $A_{eq} = \frac{G}{i} - \frac{nG}{i}\left[\frac{i}{(1+i)^n-1}\right]$

Total EUAC = EUAC1 + EUAC2

Illustrative examples – Examples 9.1 to 9.4 in Appendix A illustrate the application of the NPW and EUAC method in evaluation of alternatives in building envelope applications.

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9.5: RESERVE STUDY

A reserve study is considered a financial planning tool used by property owners, homeowners' associations, and condominium associations to assess and prepare for future major repairs and replacements of common area components. Such studies are comprehensive and go beyond façades and roofs, covering all components that are the part of the common area. This includes all common facilities and their equipment. Examples are swimming pools and recreational areas, fences and walls, irrigation and landscape equipment, roads and parking surfaces, bridges and boardwalks, fitness studios, and clubhouses, see Staebler (2021).

9.5.1: Objectives of a reserve study

9.5.2: Phases of a reserve study

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9.5.1: Objectives of a reserve study

Residential communities, including homeowners' associations (HOAs) and condominium associations, carry an implicit and continuous responsibility to safeguard the long-term functionality, safety, and sustainability of their shared properties. These communities are generally governed by an association board, which serves as the custodian of both current and future community assets. The governing board is tasked with managing resources equitably across successive generations of members – past, present, and future – while ensuring that financial obligations are distributed fairly among all the stakeholders.

A critical requirement for fulfilling this obligation is the establishment and maintenance of reserve funds. Adequate reserves influence property values in several ways and play a significant role in influencing property values for various reasons including the attractiveness of the property in the market and its positive effect on potential buyers (Staebler, 2021). In addition, some municipalities and state authorities mandate reserve studies for residential complexes, particularly those above a specified height or age threshold, to ensure that sufficient funds are available to maintain safe conditions. These requirements are especially important in contexts where rapid repair or rehabilitation may be necessary following natural events such as hurricanes, earthquakes, wild fires, or severe storms.

The overarching objectives of a reserve study are threefold:

- Financial preparedness: To ensure that adequate funds are available for the timely repair, rehabilitation, and replacement of common area components in condominiums and other multi-housing complexes.
- Safety and functionality: To maintain the property in safe condition, with building systems and components kept in proper repair and sound operational order.
- Regulatory compliance: To satisfy state or municipal mandates that require periodic funding studies, typically conducted every three to five years, depending on the jurisdiction.

From a methodological perspective, a reserve study generally comprises of two primary analyses: (1) a construction component analysis, which evaluates the condition, useful service lives, and replacement costs of major components such as façades, roofs, windows, and mechanical systems; and (2) a financial analysis, which applies principles of engineering economics to project funding needs, evaluate funding strategies, and establish long-term financial plans (Staebler, 2021).

Construction component analysis – This analysis is a physical analysis and involves a detailed inventory of all major common area components, assessment of their current condition, and estimation of their remaining useful life. As part of this analysis, the following activities are implemented:

- Inspection of assets such as roofs, façades, elevators, swimming pools, recreational areas, etc.
- Documentation of each asset's expected service life.
- Estimates of repair and replacement costs for all components.

Financial analysis – This part examines the association's current reserve fund status and develops a funding plan to ensure sufficient funds are available for future major expenditures, whether because of old components, whose service lives are exhausted, or because of unexpected natural hazards such as severe hurricanes and earthquakes. The analysis often involves the activities listed below.

- Evaluation of the current reserve fund balance.
- Development of a stable and equitable funding plan.
- Consideration of factors like inflation, interest rate, and market trends.

Since the primary purpose of a reserve study is to help property managers and owners anticipate and plan for significant expenses related to maintenance and replacement of major assets, the analysis is intended to help prevent sudden financial burdens, ensures the long-term stability of the property or association, and maintains the safe condition of the property.

Reserve studies typically cover components with a useful life of less than 30 years and provide estimates for their remaining useful life and associated maintenance or replacement costs. They are essential for effective financial planning, especially for properties requiring regular upkeep of common-area facilities.

Different kinds of reserve studies include full reserve studies, studies with site visits, and studies without site visits, each serving different purposes and conducted at varying intervals.

By providing a clear projection of future financial needs, reserve studies enable associations and property owners to make informed decisions regarding reserve contributions. This proactive planning helps maintain property values while minimizing the likelihood of unexpected special assessments or reliance on external financing, which may become costly over time.

Frequency – The frequency of conducting reserve studies can vary, but they are often recommended every 3 to 5 years to keep the information current and accurate. This frequency is primarily for newer structures and those that are less complex. However, the specific requirements and best practices can differ based on state regulations, property type, and individual owner's or association's needs. Generally, it is recommended that reserve studies be updated at least every 3 to 5 years to account for changes in property conditions, market trends, change in inflation rates, and regulatory requirements. Older and complex properties (such as high-rise structures built before 1980) may require a more stringent requirement for the frequency of a reserve study, and generally the frequency is every 2 to 3 years.

In situations when natural events such as damaging hurricanes and earthquakes occur, the reserve study may have to be done, or updated, as mandated by some states.

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9.5.2: Phases of a reserve study

A reserve study generally consists of several phases as shown in Figure 9.2 and explained below. Additional information on the phases involved in conducting the study can be found in Staebler (2021).

Pre-inspection meeting and data gathering – This step involves compiling a comprehensive inventory of all building components located in the common areas as well as those incorporated into the site improvements. At this stage, it is crucial to gather all relevant documentations related to the property, including existing architectural drawings, alteration plans, and detailed structural, mechanical, electrical, plumbing, and fire protection system drawings. Access to these documents is particularly critical when evaluating the façade and roof, as these areas are often the most costly and complex components of a reserve study.

The objective during this step is to collect detailed information on the common area assets, which includes their remaining useful life, installation date, replacement cost, quantity, and current physical conditions. Additionally, any available data regarding past inspections should be reviewed to identify prior maintenance or distress issues – such as cracks, leaks, or mechanical, electrical, and plumbing system failures – and to document the materials and methods used for previous repairs. This historical context is vital for accurately projecting future reserve needs.

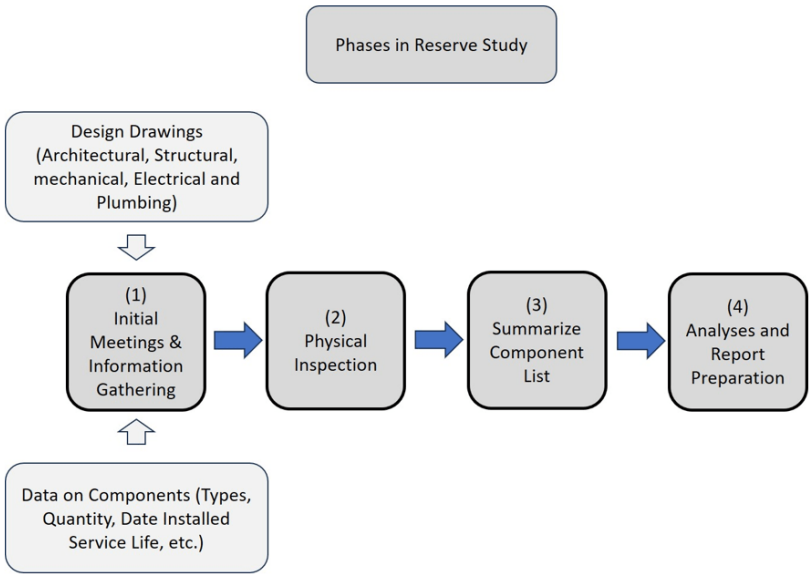


Figure 9.2 Phases in a reserve study

(This figure shows a flow chart and four steps in a reserve study starting with an initial meeting, physical inspection of the property, summarizing the components list and an analysis and report preparation.)

Consultations with the property owner are also a key aspect of this phase, ensuring that all necessary data has been collected. If the owner plans any future expansions, modifications, modernization, or alterations to the property, these plans must be thoroughly understood, as they provide critical input for refining the reserve fund strategy and ensuring that it adequately accounts for upcoming changes to the property's infrastructure.

Physical inspection – A physical inspection is conducted upon compiling all information. This is very similar to the cursory building façade inspection, described previously in this book. However, additional inspections of interior common spaces, walkways, landscapes, and various mechanical, electrical, plumbing, fire protection and security systems are also conducted as part of this phase. Depending on the property type, the inspection can last several hours during which all observations are noted, illustrations made and photographs taken. Inspections of highrises can be done on foot, but large subdivisions and mobile home parks may require transportation. Table 9.2 is adapted from Staebler (2021) and highlights an example of several important inspection points in highrise buildings. It is noted that based on the types of buildings, additional inspection points may be present in addition to those listed in Table 9.2. Inspection points related to subdivisions are also listed in Staebler (2021), to which the reader is referred.

Table 9.2 Inspection points in highrise buildings (see also Staebler, 2021)

Component Number	Description
1	Roof; decks; parapet wall
2	Chill tower; motors; fans
3	Common area heating, ventilation, and air conditioning systems
4	Elevators and their equipment and motors
5	Exterior finishes and façade elements; windows; doors
6	Stairwells and hallways
7	Railings; balconies; lanais
8	Curtain and veneer walls
9	Fire control room; fire pump; fire panel
10	Generator; transfer switches
11	Electrical and lighting system equipment
12	Pumps and their equipment
13	Plumbing system components
14	Access control; entry panel; security system
15	Interior finishes in common areas and lobby

16	Recreational rooms; swimming pools; fitness centers and their equipment
17	Lavatories; locker rooms
18	Common area meeting rooms; libraries; theaters; game and internet rooms
19	Common area decks; patios and balconies
20	Garages; access controls; motors; exhaust fans; carbon monoxide detectors

Component list summary – It is customary to summarize the compiled data in tabular form before the start of the analysis. The summary table lists the project title and the date of the reserve study. The table provides a detail of the quantity of each component, the year it was put into service, useful service life, remaining useful life and cost, see Satebler (2021) for an example of this table.

Analysis methods and report preparation – This phase involves the analysis of the data and computation of the projected funds needed for repair, rehabilitation, and replacement of the common area components and preparing a report. The owner is usually provided with at least two kinds of results. These are:

- Cash flow analysis results, which include the total reserve funding contribution amount by year for the entire period of analysis.
- Component funding analysis results, which include the total annual reserve funding contribution amount as a whole.

The computation is carried out using several available software platforms designed for capital reserve studies. However, any software selected will need to comply with the requirements of the International Capital Budgeting Institute (ICBI, 2025). Also, the reader is referred to the standards outlined by the Community Association Institute for Reserve Study, CAI (2023).

Depending on the scope and level of service, various kinds of analyses are often conducted to provide the owner or the association representative with appropriate information about planning to secure funds for future repair, rehabilitation, and replacement of common area components. Staebler (2021) outlines the kinds of model and respective computations (e.g., percent-funded calculations) that are expected to be within the capability of the software.

Three commonly-used funding analysis methods are listed below.

- Current funding method.
- Pooling funding method, which is also known as the threshold funding method.
- Component funding method, which is also known as straight-line funding model.

These methods and the computed percent-funded results are used to evaluate the financial health of the association in funding any future need for repair, rehabilitation, and replacement. Selection of a particular model depends on the funding goal decided by the owner or the association. Generally, the goal is maintaining proper repair, and keeping equipment operational with timely replacement. However, other goals may also be included in the reserve study such as any modernization, retrofit, or additions as decided by the owner or association. As expected, the current funding method is a more general procedure, which focuses on the current financial situation with the reserve. In the pooling funding method, the emphasis is on ensuring adequate reserves are always available for repairs and replacements, especially after an emergency. For example, after a major hurricane – especially in states like Florida with stricter post-disaster regulations – availability of adequate funds for repair or replacement is crucial. The pooling (threshold method) is designed specifically for this purpose, since

- It treats all reserve components as a single pool of funds.
- It maintains a minimum threshold (floor) so reserves never fall below a safe level.
- It provides flexibility to address urgent or unexpected repairs (common after disasters)

- It aligns well with regulatory goals of financial resilience and readiness.

The component funding method allocates funds separately for each component; while precise, it is less flexible in emergencies and may not ensure sufficient pooled liquidity. It is the most conservative of the three methods. Table 9.3 summarizes description of these three methods, and their limitations and applications.

Table 9.3 Description of the three funding methods employed in a reserve study

Method	Description	Goal/Application
Current Funding	Uses beginning balance (funds available at beginning of upcoming fiscal year) and a 3% annual distribution increases to estimate reserve fund	Goal: study current financial situation Application: in situations when a proper owner or association needs to know thei current financial condition
Pooling or Threshold Funding	Uses a suggested reserve distribution to maintain as threshold and a 3% annual distribution increases to estimate reserve fund	Goal: maintain the reserve balance abov certain amount in pooled funds Application: determine distribution neec to maintain threshold when owner/ association will pool reserve; also suitab for situations when repairs or replaceme after a disaster are needed
Component or Straight-Line Funding	Requires each item in component inventory to use its own account, with yearly funding; needs for timely replacements; offers a conservative way to fund reserves and results in much higher assessments per year than other two methods	Goal: determine funds needed for timely replacement Application: most commonly used in reserve study, especially when separate analyses are done for structural integrity and non-structural integrity components

Materials presented in a reserve study report – As a minimum, the reserve study report to the owner will need to include the following information, also see CAI (2023).

Information about the property, age, number of units and/or lots, physical description of the property, and current (or projected) reserve fund balance.

- A listing of the component inventory: This includes information for each component, quantities or identifying descriptions, sources utilized, useful life, remaining useful life, and current replacement cost.
- Components with long useful life (longer than 30 years): These components will need to be identified. However, as explained in CAI (2023), the funding for components beyond a 30-year useful life does not need to occur until the remaining useful life drops to 30 years.
- A description of methods and objectives employed: This includes (1) methodology used in developing the funding plan; (2) funding goals adopted in developing the plan, including the threshold goal (cash or percent funded) selected; and (3) fiscal year for which the reserve study is prepared.
- A summary description of preventive maintenance strategies as decided by the owner or the association representative: This description will need to outline the extent of preventive maintenance being employed. It is noted that in situations where preventive maintenance is not being implemented, the remaining useful life of components affected should be reduced accordingly.
- A summary of any discussions and/or plans about periodic structural inspection reports: Periodic structural inspection reports are important documents and will need to be reviewed so that any corrective maintenance or repair conducted are

properly included in the reserve study with information about the respective timing and costs.
- A description of the scope and level of service and the limitation of the reserve study: In many cases, the scope of service limits to conducting the reserve study for (1) structural integrity components only; and (2) all other components. Results for these different analyses will need to be properly explained in the report.

It is noted that there are variations to the above list of sections of the report. Furthermore, the report will properly need to include illustrations and photographs identifying areas and components visited and inspected at the time of the physical examination.

Illustrative Examples – Example 9.5 in Appendix A demonstrates the applicability of the component (straight line) method in a façade component replacement. Additional examples highlighting the applicability of the three aforementioned fund methods can be found in Staebler (2021).

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9.6: Chapter References

CAI (2023). *Reserve study standards*, published by Community Association Institute, Falls Chrch, Virginia

ICBI (2025). *Software certification*, published by International Capital Budgeting Institute, Washington, District of Columbia (www.capitalbudgeting.org).

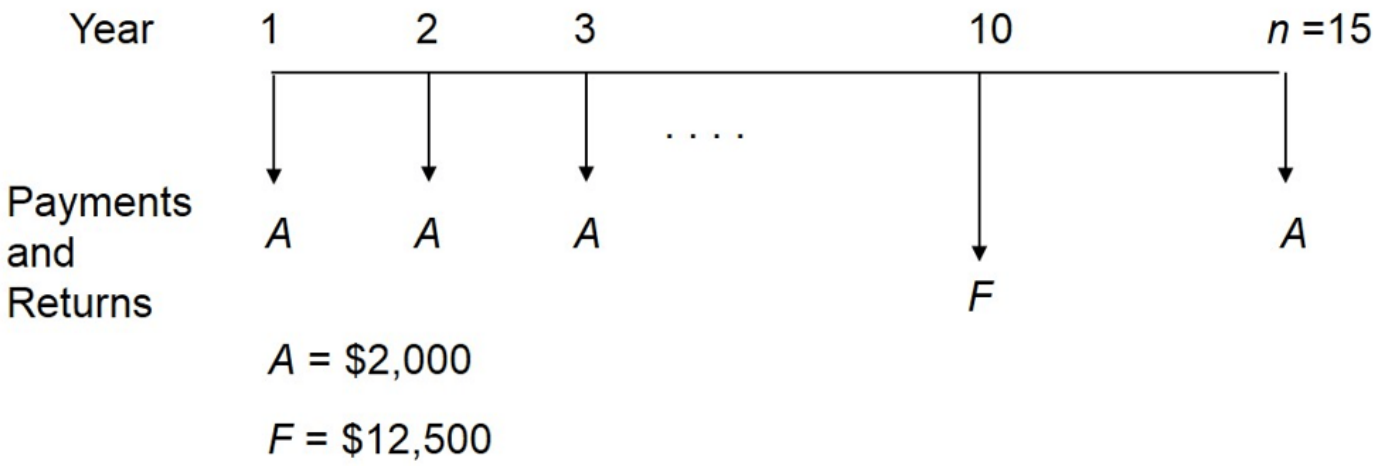
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9.7: Exercises

E9.1 The accompanying timeline is a representative of an option in delaying the needed repair for façades of a building. Using an interest rate of 4% per year and a life of n equal to 15 years, compute the equivalent uniform annual cost (EUAC) for this option.



(A time line of 15 years is shown in this figure with equal uniform payments and a future cost in year 10.)

E9.2 Two options are suggested for the rehabilitation of the envelope in a building. For this problem n is equal to 20 years and the interest rate i is 4% per year.

Option 1: Do nothing, yearly maintenance and repair is $6,000 in the first year; and this will increase every year by $500 starting in the second year; penalty paid to the city: in year 10 and year 20 only, each $4,000.

Option 2: Pay for the entire cost of rehabilitation now for $95,000. There is a yearly maintenance cost of $1,000. However, there is a rebate of $4,000 from the power company in year 5.

Compute the net present worth for each option and decide which option is more favorable.

E9.3 Currently, it costs $980,000 to replace all common area windows and doors in a residential complex. Compute the future worth of this value for a 10-year service life and an inflation rate of 3% per year.

E9.4 In Problem described in E9.3, compute the equivalent annual unform cost.

E9.5 Which of the following statements is most nearly correct?

a) An engineering economics study is always a part of a building façade inspection project.

b) An engineering economics study is never a part of a building façade inspection project.

c) An engineering economics study is sometimes a part of a building façade inspection project.

d) A façade inspection project always is accompanied by a reserve study.

E9.6 Which of the following statements is most nearly correct?

a) A reserve study always includes a physical inspection of the property.

b) A reserve study sometimes includes a physical inspection of the property.

c) A reserve study never includes a physical inspection of the property.

d) A reserve study is always included in a façade inspection report.

E9.7 A condominium association is planning to conduct a reserve study as part of the state requirement following a major hurricane. The objective is to have a comprehensive plan for maintaining a minimum level of funds available for timely component repair, rehabilitation and replacement and also include a structural integrity study. Which of the following statements is most nearly accurate in selecting the funding method?

a) The current funding method.

b) The pooling (threshold) funding method.

c) The component (straight line method).

d) None of the three methods can be used.

E9.8 Which of the following statements is most nearly accurate?

a) A 3-year frequency for a reserve study is generally used.

b) A short than 3-year frequency may be necessary for preparing a reserve study for older structures.

c) An update of a reserve study may be needed when there is a severe natural hazard such as a hurricane or earthquake in the area.

d) All of the above.

E9.9 Which of the following statements best describes the purpose of a reserve study?

a) It is needed to as part of routine ongoing façade inspection reports.

b) It is needed to ensure adequate funds are maintained for timely repair, rehabilitation and replacement of common areas in multi-unit buildings.

c) It is needed to only meet the requirements by insurance companies.

d) It is needed in any real estate transaction in condominiums.

E9.10 A complete tuckpointing and façade repair on the exterior façades of a condominium association costs 1.2 million dollars. The association plans to do this work in 10 years and start a special assessment for the work. Conduct a component (straight-line) funding analysis to list yearly investment needed for timely repair of the façade as planned. Use 3.4% inflation rate. The association will invest the distribution in a saving account resulting an interest rate of 1.7% per year. Currently, there are no funds allocated to the façade repair. The estimated cost for minor repair, painting and maintenance is $8,000 in year one and increases by 3% every year. Note that per industry standards, the yearly assessment fee collected will increase by 3% every year.

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10: USE OF VR/AR TECHNOLOGIES IN BUILDING ENVELOPE INSPECTION

Chapter Synopsis

This chapter introduces the emerging applications of Virtual Reality (VR) and Augmented Reality (AR) technologies in the field of building envelope inspection, damage assessment, and repair planning. It explores how VR and AR tools allow inspectors, engineers, and students to visualize both visible and hidden areas of building envelopes, enhancing understanding and detection of potential damage that may otherwise be difficult to observe.

The chapter also integrates the use of 3D modeling and 3D printing technologies to replicate façade components such as brick, stone, or terra-cotta blocks, including hidden support structures and connections. These hands-on activities, derived from 2D inspection images, enable students to deepen their understanding of façade construction and failure mechanisms.

The chapter aligns with the broader objective of the book as an Open Educational Resource (OER) by providing accessible learning methods, interactive tools, and practical exercises that can be incorporated into architectural, engineering, and construction education.

10.1: Introductory Remarks

10.2: Fundamentals of VR, AR, and MR Technologies

10.3: 3D Visualization of Building Envelope Damage

10.4: Application of VR/AR/MR in Field Inspection

10.5: Damage Assessment and Repair Planning with VR/AR

10.6: Spatial visualization of building envelope details

10.7: Integrating 3D Printing for Educational Exercises

10.8: Chapter References

10.9: Exercises

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10.1: Introductory Remarks

Importance of advanced technologies in building envelope inspection – As urban municipalities increasingly mandate periodic building envelope inspections for structures exceeding certain height thresholds, the industry has likewise evolved to adopt more advanced technologies to meet these regulatory demands. The primary goal is to reduce the labor intensity and time required for on-site inspections while simultaneously achieving greater coverage and accuracy in assessing façades and roofing systems.

One notable advancement is the use of drones, which allow inspectors to access and document difficult-to-reach areas efficiently and safely. Drones equipped with high-resolution cameras can capture detailed imagery of building exteriors without the need for costly scaffolding or lifts. Similarly, laser-based scanning technologies, such as LiDAR, have enabled engineers and architects to develop highly accurate three-dimensional models of building envelopes in a fraction of the time traditionally required for manual surveys.

While these techniques are particularly effective for inspecting the visible exterior surfaces, challenges remain when investigating areas that are concealed behind façade elements. Hidden damages often require more intrusive methods to detect and evaluate. Currently, standard practice involves physically removing sections of the façade to gain access to the underlying structures for a better understanding of the type and extent of damage present. For instance, as discussed in Chapter 4, hidden issues such as corroded or missing connection rods – critical components that secure terra-cotta or stone blocks to the building's structural system – may only become apparent through such invasive investigation.

However, there are practical limitations to how many blocks can be removed without compromising the façade's integrity or incurring significant costs. As a result, a high degree of uncertainty often persists regarding the overall condition of the envelope.

In this context, advanced visualization technologies offer promising solutions. The development of detailed 3D models of façade elements, including the conditions behind them, can significantly enhance understanding of potential damage without extensive removal of materials. Specifically, the application of Virtual Reality (VR). Augmented Reality (AR) and Mixed Reality (MR) technologies presents a powerful means to simulate and visualize hidden conditions, thereby improving diagnostic capabilities and informing targeted repair strategies. By integrating VR and AR tools into the inspection process, engineers and architects can achieve a more comprehensive assessment while minimizing disruption to the building envelope.

These technologies not only increase efficiency and safety, but also contribute to better long-term maintenance planning, supporting the overarching goals of durability, sustainability, and cost-effectiveness in building envelope management.

Potential application of VR and AR in the construction and maintenance industry – The VR and AR applications offer transformative technologies that enhance graphics visualization by immersing users in interactive, three-dimensional environments. VR creates fully immersive digital worlds, enabling users to experience and interact with simulated spaces, while AR overlays digital information onto the real world, enhancing the user's perception of their environment.

Despite growing recognition of their potential, the application of AR and VR technologies in the construction industry remains limited. Davila Delgado, Oyedele, Beach, and Demian (2020) analyze factors that either hinder or promote the adoption of these technologies. Ahn, Han, and Al-Hussein (2019) demonstrate a proof of concept using 2D drawing visualization for projection-based AR in a panelized construction manufacturing facility. Although widespread adoption has yet to occur, the visualization capabilities of AR and VR are acknowledged for their value in enhancing design processes, training, and operational efficiency in construction and maintenance.

Specifically, the VR/AR technologies are expected to evolve as powerful tools in project planning and execution. VR allows architects and engineers to visualize and interact with 3D models of buildings before construction begins, facilitating better design decisions and early detection of potential issues. AR, on the other hand, enables workers on-site to overlay digital blueprints onto physical structures, aiding in accurate installation and real-time progress tracking.

These technologies also play a crucial role in maintenance operations. AR can provide technicians with real-time, hands-free access to equipment manuals and diagnostic information, enhancing repair accuracy and reducing downtime. Additionally, VR is used for safety training, allowing workers to experience hazardous scenarios in a controlled virtual environment, thereby improving preparedness without exposure to real-world risks.

Purpose and educational value within an Open Educational Resource (OER) framework – Aligned with the principles of Open Educational Resources (OER), this resource aims to provide an interactive learning tool that leverages VR/AR technologies to enhance the understanding and visualization of damage scenarios in building envelopes and their components – conditions that are often abstract and difficult to conceptualize through traditional means. By employing these immersive technologies, learners are able to explore multiple repair and treatment alternatives for damaged areas. This facilitates a more informed evaluation of each approach, supporting the selection of the most durable and cost-effective solution.

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10.2: Fundamentals of VR, AR, and MR Technologies

As discussed in the previous section, VR, AR, and MR are immersive technologies designed to enhance human perception and interaction with digital content. While these technologies are grounded in similar foundational principles, they differ substantially in how users engage with virtual and physical environments. This section examines VR, AR, and MR in greater detail by defining their respective application types and highlighting the key distinctions among them.

VR enables the creation of entirely computer-generated environments that give users the sensation of being completely immersed within a virtual environment. VR replaces the real-world view with a simulated 3D environment, typically experience through head-mounted displays, controllers, and motion-tracking systems. In the context of building envelope studies, VR allows users to explore digital twins or reconstructed models of building exteriors and interiors, enabling visualization of structural systems, envelope components, and potential failure points without physical constraints.

AR, by contrast, overlays digital information (e.g., annotations, 3D models, sensor data, etc.) onto the real-world environment view. This integration enhances, rather than replaces, physical perception. AR is particularly valuable for on-site building envelope inspections, as inspectors can visualize hidden components, defect locations, or material property data directly within their field of view using AR glasses, tablets, or mobile devices.

MR extends AR by enabling real and virtual objects to interact dynamically in real time. For example, an MR application may allow an inspector to manipulate a digital model of a window system or façade joint while observing its alignment with the actual construction. Thus, MR bridges the gap between VR's fully virtual environment and AR's real-world augmentation, offering interactive and context-aware analysis capabilities.

Figure 10.1 illustrates the virtual-reality continuum with example images of VR, AR, and MR. The extreme ends of the spectrum represent the real and fully virtual environments, while the intermediate examples demonstrate how the virtual object, such as a building image, is integrated differently across VR, AR, and MR environments.

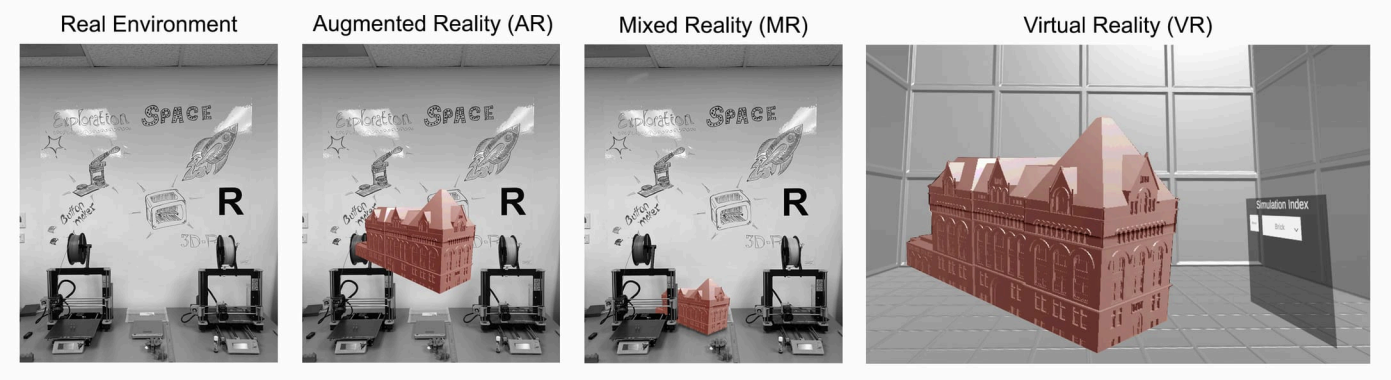


Figure 10.1 Difference between AR, MR, and VR (photo of real environment is from personal collection of the authors).

(This figure shows four images. The left image depicts a real environment in Illinois Tech's Exploration Space at Galvin Library, featuring a table with two 3D printers. The second image shows a model of Illinois Tech's Main Building overlaid onto the real environment as augmented reality. The third image presents the same digital model integrated into the physical space as mixed reality. The final image shows an immersive virtual reality view of the building, fully detached from the physical environment.)

Immersive technologies rely on integrated hardware and software ecosystems to capture, process, and visualize complex spatial information. Hardware components commonly used in VR, AR, and MR applications include head-mounted displays, mobile and tablet devices, 3D scanners and LiDAR sensors, cameras, drones, etc. These devices support immersive visualization as well as on-site data capture, overlay, and annotation during building envelope inspection. Software tools used in VR/AR-based building envelope applications include 3D modeling and Building Information Modeling (BIM) platform (e.g., Autodesk Revit, Rhino, and SketchUp), visualization platforms, AR/VR integration tools (e.g., Autodesk XR, Dassault Systèmes' 3DEXPERIENCE), data management and cloud-based Systems, etc. Together, these tools enable detailed representation of envelope systems and provide real-time rendering, interaction, and simulation environments.

The VR and AR technologies enable inspectors, designers, and contractors to intuitively explore building envelopes in three dimensions, improving understanding of geometry, interfaces, and material transitions. By visualizing assemblies and connections at full scale, potential constructability issues or design conflicts can be identified earlier in the project process. In particular, AR can further support field inspections by guiding inspectors through complex inspection procedures and overlaying relevant data such as component specifications, moisture readings, and maintenance histories. The MR technology further enhances real-time interaction with digital models, promoting more effective collaboration and informed decision-making among architects, engineers, and building scientists.

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10.3: 3D Visualization of Building Envelope Damage

Accurate visualization of damage in building envelope systems is essential for assessing deterioration, planning maintenance, and developing effective repair strategies. Traditional inspection approaches rely heavily on 2D images, sketches, or field notes, which often lack spatial context and make it difficult to fully understand the extent and depth of damage. In contrast, 3D visualization techniques enable the transformation of inspection data into spatially accurate digital models that reveal both visible and concealed defects. These methods enhance diagnostic accuracy, support data-driven decision-making, and facilitate communication among engineers, architects, inspectors, and facility managers.

Figure 1.14 (in Chapter 1) presents a sketch-drawing illustrating the attachment of terra-cotta blocks to a masonry wall. Without prior familiarity with terra-cotta anchorage systems, the details shown in this drawing may appear abstract and difficult to interpret, particularly for students. Figure 10.2a presents a three-dimensional representation in which the terra-cotta blocks are visually separated from the supporting masonry wall. One component that is especially challenging to convey in a two-dimensional representation is the presence and location of internal stiffening elements within each block, commonly referred to as *webbings*. To address these limitations, A block is isolated from the rest of the assembly and shown in the 3D image of Figure 10.2b on the left. The detail is further enhanced when an AI-generated photo-like image is produced and shown to the right in Figure 10.2b. The 3D detail and the AI-generated image further clarify the locations of the stiffening webbings and the way the anchor system is installed for attaching the block to the support structural system.

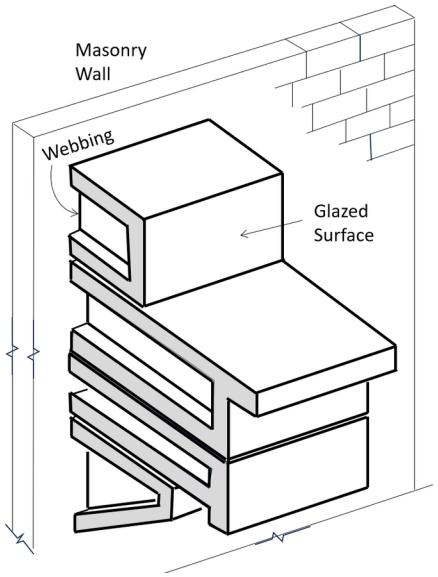


Figure 10.2a 3D drawing of terra-cotta blocks and the masonry wall, developed from the 2D detail of Figure 1.14

(This figure shows a 3D drawing of terra-cotta blocks attached to a masonry wall. In this detail blocks are separated from the wall to enhance understanding of the block shapes with a glazed surface in front and open in the back.)

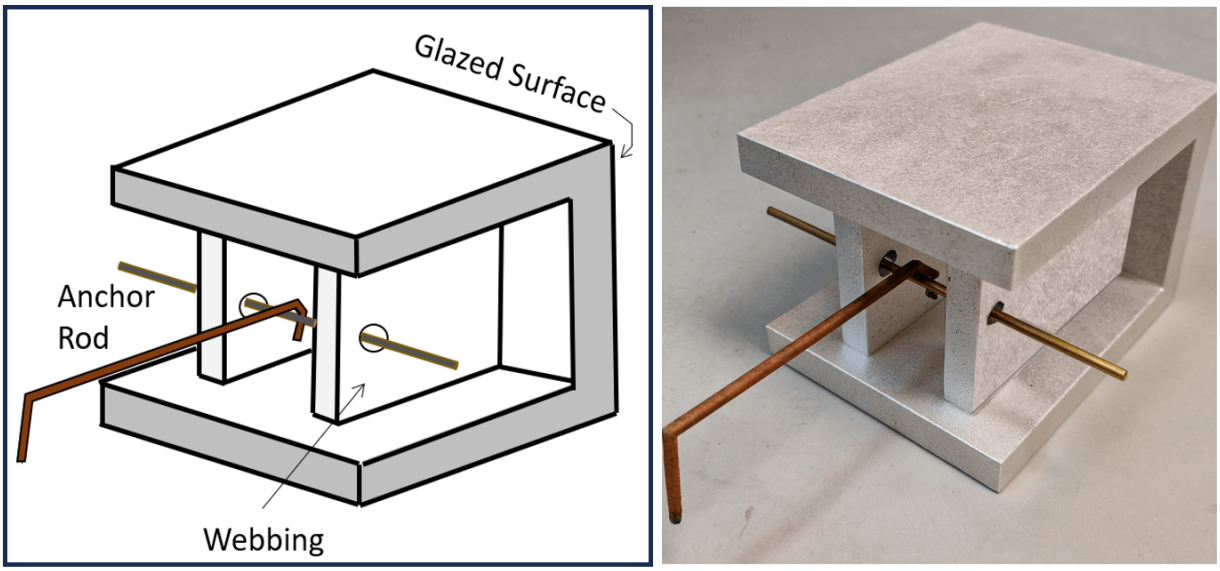


Figure 10.2b 3D detail of a terra-cotta block with anchorage system and the AI-generated realstic photo-like image of the block

(This figure shows a 3D detail of a terra-cotta block and the anchorage rod assembly on the left. The detail shows a view from the back end of the block. To the right a photo-like image of the block is shown as generated by the AI.)

The generation of the three-dimensional visualization can easily be achieved by the AI, as well as the representation of any observed damage to the anchors or terra-cotta units, can be achieved primarily through a textual description of the condition or through the input of a hand-drawn sketch of the detail. This approach enhances the interpretability of traditional construction documents and supports improved understanding of façade attachment systems, particularly for inspection, assessment, and training purposes.

Developing reliable 3D representations of building envelope damage typically begins with the collection of high-resolution 2D datasets, such as photographs, thermal images, LiDAR scans, and inspection drawings, which are then converted into cohesive spatial models. Several techniques are used to achieve this transformation, including photogrammetry, laser scanning, integration of 2D inspection results within BIM or CAD platforms, thermal and multi-spectral mapping, and so on.

2D inspection results, such as annotated photographs, material test results, or infrared images, can be imported and spatially referenced within BIM or CAD platforms. Through this process, damage information is geolocated within a digital building model, allowing engineers to associate specific defects with corresponding building components. For example, crack patterns identified in field images can be mapped onto a 3D façade model within a BIM environment, facilitating trend analysis and comparison across multiple inspection periods.

In the following sections, several methods currently employed in building façade inspections are reviewed, and their integration with 3D visualization is discussed as a potential approach for more effectively representing building envelope damage scenarios.

Multispectral infrared thermography – This technique provides a non-destructive means of detecting anomalies related to moisture intrusion, delamination, or thermal bridging (Tomita and Chew, 2022). By converting these 2D thermal patterns into 3D heat-mapped surfaces, inspectors can visualize temperature gradients across façades, revealing subsurface damage conditions such as water ingress behind cladding or insulation voids.

Photogrammetry – This technique involves processing multiple overlapping 2D photographs to generate 3D point clouds and textured surface meshes. Using appropriate software, this technique estimates depth and geometry by matching visual features across multiple images captured from different viewpoints (Kinoshita et al., 2024). When applied to façade inspections, photogrammetry can recreate accurate 3D models of exterior walls, window systems, or roof assemblies. These reconstructions capture surface irregularities with millimeter-level precision, making them valuable for visual damage mapping, such as cracking, displacement, material loss, etc.

LiDAR – This technology allows generating dense point clouds that accurately represent the geometry of building envelopes. Unlike photogrammetry, which depends on visible texture, LiDAR directly measures surface distances using reflected laser pulses.

This capability allows reliable modeling of structures with limited visual contrast, such as uniform cladding panels or reflective glass façades.

LiDAR datasets can be merged with high-resolution imagery or infrared thermography to produce hybrid 3D models that capture both geometry and surface conditions, enabling comprehensive visualization of damage zones (Catbas et al., 2022).

3D visualization technologies allow better, and a clearer, representation of a damage scenario in a building envelope by converting 2D inspection data into spatially coherent digital representations. Through the integration of these technologies, inspectors will be able to visualize both visible and hidden deterioration in ways that were previously unattainable. This capability enables simultaneous examination of surface-level damage and hidden or subsurface defects, improving the accuracy of building envelope assessments.

Visible damage, including cracks, spalling, efflorescence, corrosion, and material deformation, can be represented in 3D models using texture mapping or geometric deformation techniques that replicate the size, depth, and pattern of surface deterioration. For example, a photogrammetric model of a concrete façade can highlight crack propagation paths in color-coded overlays, while localized mesh displacement can represent depth variations caused by surface erosion or blistering coatings.

Hidden damage scenarios, such as delamination, corrosion beneath cladding, voids within insulation layers, or bond failures, is often not detectable through visual inspection alone. Integrating non-destructive evaluation (NDE) data into 3D visualization frameworks allows these concealed defects to be spatially represented within the digital model.

Thermographic mapping – This technique visualizes heat anomalies indicative of trapped moisture, air pockets, delamination, and so on. Tomita and Chew (2022) provide an example of delamination in a concealed building wall detected using infrared imaging, where surface temperatures above delaminated areas are higher than those of surrounding regions. Such defects appear as positive thermal contrasts or hot spots in infrared images.

Ultrasonic inspection – Data compiled through this method provide quantitative measurements and qualitative images of a structure's internal condition, revealing hidden defects such as cracks, voids, corrosion, etc. When embedded as internal defect layers within the 3D envelope model, ultrasonic data enable volumetric visualization of subsurface deterioration. These visualization layers help correlate observed surface conditions with internal damage mechanisms, supporting more informed assessment and prioritization of repair strategies.

- 3D visualization of building envelope damage scenarios provides several practical and analytical benefits, as described below.
- Facilitates the identification and classification of deterioration across the entire façade and roof.
- Enables volumetric and surface area calculations of damaged regions, supporting more precise repair cost estimation.
- Allows comparison of successive inspection datasets to track deterioration progression over time.
- Offers intuitive, data-rich visualization that assist stakeholders in prioritizing maintenance actions and validating remedial designs.
- Serves as an effective educational tool for inspectors and as a permanent visual record of the building envelope condition at specific inspection intervals.

The ability to accurately depict and analyze cracks, delamination, corrosion, and other forms of damage scenarios within immersive and data-rich environments enhances understanding of failure mechanisms and supports more effective maintenance strategies. As sensing technologies and computational tools continue to advance, 3D visualization is expected to play an increasingly important role in data-driven building envelope diagnostics and management.

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10.4: Application of VR/AR/MR in Field Inspection

Field inspection of building envelopes, particularly façades and roof systems, poses significant challenges due to limited accessibility, complex geometries, safety constraints, and so on. Conventional inspection methods rely heavily on visual observation, photography, and manual note-taking, which can result in insufficient or incomplete documentation and subjective interpretation of defects. The use of VR, AR, and MR technologies offers a more advanced and efficient approach to field inspection by enhancing situational awareness, improving data accuracy, and enabling safer and more effective evaluation of building envelopes. These immersive technologies integrate real-time data visualization with spatial digital models, allowing inspectors to analyze, annotate, and communicate inspection findings within interactive 3D environments.

For example, once 3D damage models are developed, they can be imported into VR, AR, or MR platforms to enhance the interpretation of potential damage areas within a part or the entire façade. In a VR platform, inspectors can virtually navigate a damaged façade, zoom in on cracks, or measure areas affected by corrosion at full scale. When using AR on site, digital 3D damage overlays can be projected onto the physical building surface, allowing direct comparison between the digital diagnosis and actual conditions. MR further supports interactive exploration by enabling users to investigate cladding layers virtually and visualize hidden corrosion or delamination within their real spatial context. This integration of diagnostic data with physical inspection workflows improves coordination and communication among designers, engineers, and maintenance personnel.

During on-site inspections, AR and MR play a critical role by superimposing digital information directly onto the physical façade or envelope surface. Using AR/MR-enabled devices, inspectors can view BIM models, damage maps, and sensor data accurately aligned with the real-world structure. For example, AR or MR can support design-to-as-built comparison by displaying deviations between the intended BIM geometry and actual construction. Defect overlays, such as cracks, moisture intrusion, or corrosion identified from previous 3D scans, can be projected onto the real surface for verification. These capabilities reduce ambiguity in locating and classifying defects, minimize the need for manual reference to drawings, and enhance consistency in field data collection. Inspectors can also record annotations, photographs, or voice notes directly within the AR interface, automatically linking observations to their spatial locations.

VR enables immersive exploration of digital replicas or digital twins of building envelopes without requiring physical site presence. Using 3D models generated from photogrammetry, LiDAR scans, or BIM data, inspectors and project stakeholders can conduct virtual site walkthroughs in simulated environments that closely replicate real conditions. For pre-inspection planning, VR environments allow project teams to study façade geometry, access limitations, and safety zones before deploying personnel or equipment. During inspections, remote experts can participate by sharing synchronized 3D models or real-time video streams with on-site teams, supporting inspection tasks and verification of findings. In addition, historical inspection data can be embedded within virtual environments for temporal comparison or used as training modules for new inspectors learning to identify façade defects. These approaches significantly improve inspection efficiency, particularly for large-scale projects where access to all areas of the façade is limited.

VR, AR, and MR technologies also provide innovative solutions for overcoming logistical, accessibility, and safety challenges associated with building envelope inspections, such as tall façades in high-rise buildings, intricate geometries, inaccessible envelope components, etc. In practice, engineering consulting firms have already begun adopting VR immersive design reviews and AR for on-site visualization of constructed or planned building elements. VR enables clients and project teams to virtually walk through and collaborate on building designs prior to construction, while AR supports tasks such as overlaying 3D models onto physical structures for construction visualization or finish evaluation.

In addition to AR/VR technologies, artificial intelligence (AI)-driven platforms have also been employed by some firms to analyzes photographs and videos captured by mobile devices and drones to automatically detect, classify, and catalog damage to building façades and structural components. Such platforms illustrate the growing integration of AI with immersive and digital technologies to enhance inspection accuracy, efficiency, and decision-making in building envelope management.

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10.5: Damage Assessment and Repair Planning with VR/AR

In the life-cycle management of building envelopes, damage assessment and repair planning are critical stages that require accurate diagnosis and effective prioritization. Traditional assessment methods rely primarily on close or afar examination of damaged areas, 2D sketches, and photographic documentation. These approaches are often time-consuming and may lack the spatial and temporal resolution necessary to fully understand damage evolution or to evaluate the long-term implications of repair strategies.

Over the past decade, extensive research has investigated novel methods for condition assessment and structural health monitoring. Although not widely used in building envelope condition assessment, various automated implementations have emerged, including the use of MR to assist the field engineer in better understanding damage areas, and VR to bring the field-collected data into office environments. These techniques have afforded multiple stakeholders collaborate within a shared virtual space (Catbas et al., 2022). Emerging VR and AR technologies provide powerful tools for enhancing damage assessment and repair planning by enabling immersive visualization, interactive simulation, and predictive modeling of damage conditions and remediation scenarios. By integrating digital twins, sensor data, and inspection findings into immersive environments, professionals can not only visualize existing damage but also simulate its progression and evaluate potential repair interventions and to certain extent, even predict the growth of damage. This is especially true when damage, for example a crack appearing on walls, is dynamic and affected by a host of environmental and site factors, which require either long-term monitoring or advanced analytical modeling.

As discussed earlier, VR and AR technologies allow engineers and inspectors to visualize and simulate how damage in building envelopes evolves over time and how various repair actions influence long-term performance. Using 3D models derived from inspection data, such as photogrammetry or LiDAR scans, damage phenomena can be simulated within virtual environments, e.g., crack propagation, corrosion of metallic components, or moisture-induced delamination. In VR, inspectors and engineers can observe the temporal evolution of degradation under different environmental or loading conditions, such as (1) the expansion of cracks in masonry or concrete façades due to thermal cycling, (2) the spread of corrosion beneath metal cladding in humid or coastal environments, and (3) the migration of moisture through insulation and vapor barrier layers leading to condensation or mold growth. These visualizations provide intuitive insight into deterioration mechanisms and support early identification of critical areas requiring intervention in building envelope damage assessments.

VR platforms also enable engineers to test repair strategies in a virtual space prior to physical execution. Through interactive 3D modeling, engineers can simulate the replacement of façade panels or the installation of waterproofing membranes. Possible various material performance parameters, such as adhesion, permeability, and thermal resistance, can be incorporated into simulations to visualize the effects of different repair options on overall envelope behavior. AR can complement this process by enabling on-site projection of repair sequences directly onto the affected surfaces. For example, an inspector using an AR headset may view step-by-step visual guides for injecting epoxy into cracks or reinstalling damaged insulation boards, ensuring precision and procedural compliance during repair fulfillment. These capabilities allow to significantly reduce trial-and-error in the field and support more informed decision-making regarding material selection and repair prioritization.

In civil structure applications such as bridge engineering, structural health monitoring has been widely adopted to overcome limitations of traditional methods, including high labor demands, time consumption, and uncertainty and bias inherent in condition rating assigned to structural members by inspectors. The modern structural heath monitoring techniques allow assessing the condition of structures, primarily large-scale and, in some cases, medium-scale systems, by collecting sensor data using instrumentations such as accelerometers, strain gauges, potentiometers, cameras, lasers, as well as NDE techniques (Catbas et al., 2022). For building envelope condition assessment, similar sensor data can be collected periodically or continuously throughout a building's life cycle. Analysis of these sensor datasets, combined with historical inspection records, enables integrated damage diagnosis and prognosis.

Furthermore, the integration of VR and AR technologies with sensor data enables the development of predictive damage models, representing a significant advancement in condition-based maintenance of building envelopes. By integrating real-time inspection data with computational models and AI-driven analytics, AR systems can generate interactive forecasts of future damage states. Algorithms trained on historical inspection records and sensor datasets can predict potential defect initiation, damage evolution, and deterioration progression. These predicted damage states can then be visualized through AR interfaces directly on the physical building envelope, enabling inspectors and facility managers to anticipate deterioration before it becomes visually detectable.

For example, an AR system may project a heatmap onto a glass curtain wall to highlight areas of elevated thermal stress that are predicted to lead to sealant failure. Glass panels are particularly susceptible to thermally induced stress breakage, which occurs when temperature differentials develop across the glass surface. These differentials are often caused by non-uniform solar radiation exposure, resulting in temperature gradients that generate tensile stresses and may ultimately lead to fracture (Mohammadi et al., 2022).

A heatmap overlay applied to a glass façade can support the inspection process by providing localized, real-time information on façade regions and glass panels that are more vulnerable to damage. This visualization enables engineers to prioritize and focus visual inspections on high-risk areas. Furthermore, such a proactive assessment approach allows maintenance teams to plan targeted interventions, thereby reducing the likelihood of costly and disruptive repairs later in the building life cycle.

Building envelope repair planning based on the condition assessments and predicted damage states can be significantly enhanced through VR and AR technologies. These tools are particularly effective for supporting decision-makers, such as building owners, architects, and contractors, by improving their understanding of the technical and financial implications of building envelope deterioration, inspection findings, proposed repair strategies, and also communicate each other. Through VR-based review sessions, decision-makers can explore immersive models of the damaged envelope systems, viewing areas of concern at realistic scale and within their spatial context. As these technologies continue to integrate with digital twin platforms and AI-driven analytics, the future of building envelope assessment ae expected to shift toward continuous and intelligent asset management approaches.

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10.6: Spatial visualization of building envelope details

A challenge in engineering instruction is enhancing students' ability to visualize three-dimensional objects from two-dimensional representations and, conversely, to accurately translate spatial ideas into orthographic drawings. For this reason, engineering programs include foundational courses in engineering graphics to develop spatial visualization skills and build competency in blueprint reading. *Introduction to Engineering Drawing and Design*, developed for first-year students in engineering and architecture, introduces technical drawing and orthographic projection as essential tools for communicating engineering designs, structures, and systems.

Similar challenges arise in the interpretation of building envelope sections. Building envelopes details are typically represented through vertical and horizontal section views that depict multi-layered assemblies in two dimensions. These representations can appear highly abstract, and understanding how the different layers are assembled and integrated often demands exposure to hands-on, practical experience that traditional coursework does not always provide.

Sections 10.6 and 10.7 investigate the role of three-dimensional representation and advanced visualization tools – including augmented reality (AR) and virtual reality (VR) – together with physical fabrication methods such as 3D printing, in supporting the comprehension of building sections and envelope design. These advanced visualization and fabrication techniques overcome the limitations of two-dimensional representation by allowing users to rotate, explode, and deconstruct models, thereby providing an enhanced three-dimensional learning experience.

Section 10.6 walks through a building envelope detail to provide an in depth understanding of the assembly and its representation. Section 10.7 presents a hands-on 3D printing exercise that can be replicated by readers or implemented in the classroom.

Example of envelope detail: Brick rain screen with CMU blocks – The example of a brick rain screen wall with CMU block backing is used to illustrate the challenges posed by 2D detail visualization. A typical wall detail looks like the image below. This detail includes a series of lines and hatches representing the different layers and materials of the wall assembly in the section view. it can appear abstract and difficult to interpret for first-year architecture and engineering students who often lack practical field experience.

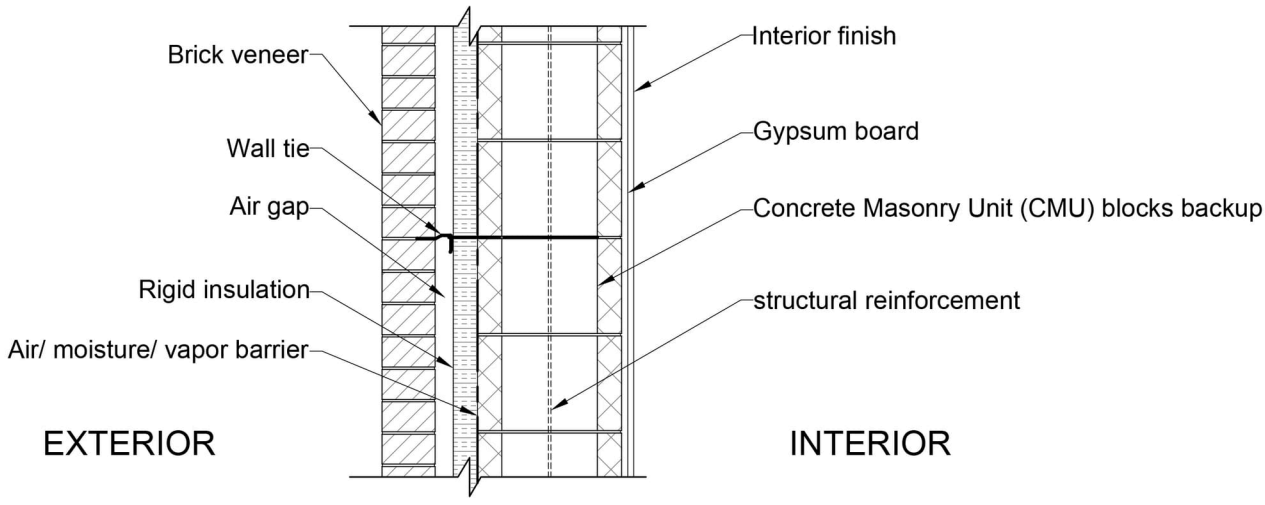


Figure 10.3 Building envelope detail: Brick rain screen with CMU blocks

(Figure 10.3. represents a vertical section detail of a brick rainscreen wall assembly with a CMU backup wall. Labels are included to identify the individual elements that comprise the wall assembly.)

Figure 10.3 represents a vertical section cut through a wall assembly showing the different layers making up the assembly. The

images below help explain how the wall detail above is created.

Figure 10.4a illustrates an imaginary cutting plane passing through a wall, shown in blue. If we imagine removing the portion of the wall located to the right of the cutting plane, the resulting view reveals the internal layers of the wall assembly (Figure 10.4b).

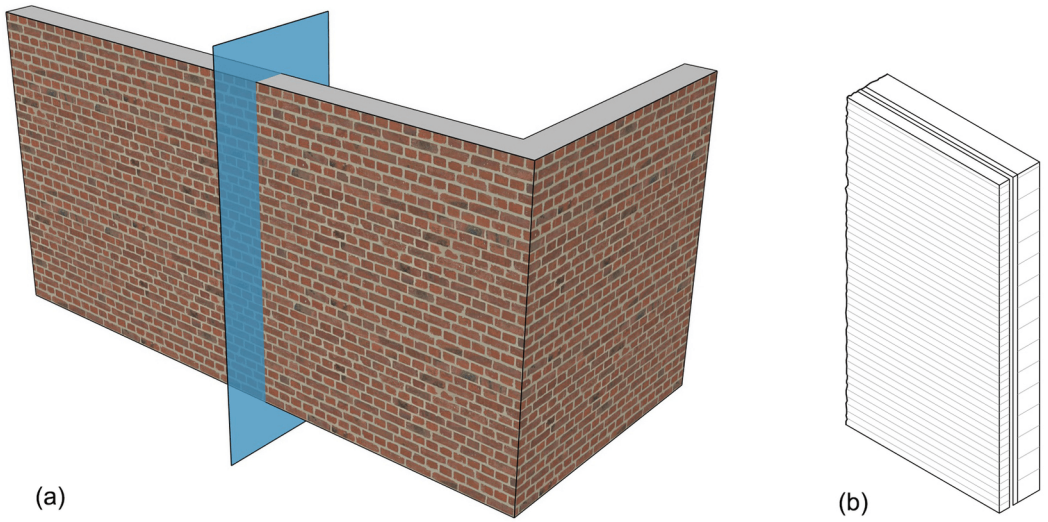


Figure 10.4a Imaginary plane cutting through a masonry wall; Figure 10.4b 3D view of sectioned wall

(These figures illustrate the creation of a vertical section through a wall assembly. A cutting plane passes through the wall, allowing the viewer to look into the assembly and see the individual layers and components that make up the wall system.)

A wall assembly is composed of multiple layers, each serving a different function. These layers can generally be grouped into three main categories: cladding, control layers, and structure.

- **Cladding** refers to the exterior protective layer of the wall. In this example, the brick veneer acts as the cladding and helps protect the building from environmental exposure.
- **Control layers** regulate the movement of heat, air, and moisture through the wall assembly. These layers may include rigid thermal insulation, air barriers, moisture barriers, and vapor barriers.
- **Structural components** provide support and stability to the building. In this case, the CMU (concrete masonry unit) blocks form the primary structural system of the wall.

The diagram below (Figure 10.5) represents the “perfect wall,” with the different layers highlighted (Lstiburek, 2010).

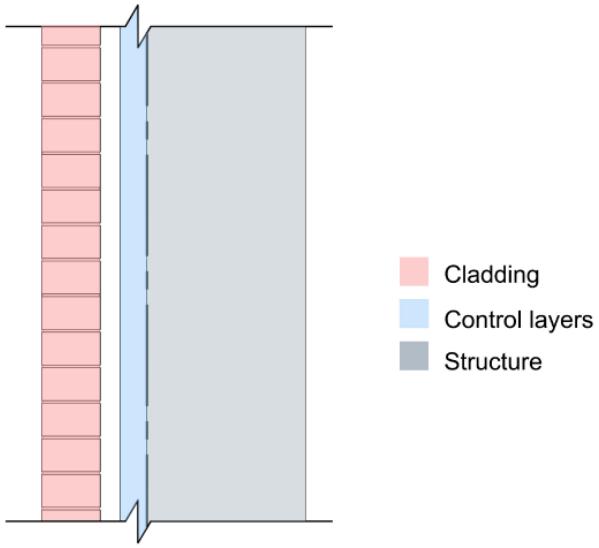


Figure 10.5 Diagram of the "perfect wall"

(This figure shows a diagram of the "perfect wall," identifying the structural system, control layers, and cladding.)

Even when the layers composing the wall assembly are clearly represented, it can still be difficult to understand how these elements are assembled in three dimensions from a vertical section alone. Without the support of an accompanying horizontal section or a three-dimensional representation, important spatial relationships and connections between components may remain unclear. For example, the hidden line running through the CMU blocks represents the structural reinforcement embedded within the wall, while the brick veneer is connected to the backup wall through metal wall ties.

The image below (Figure 10.6) shows the wall detail with the wall ties highlighted, followed by two three-dimensional visualizations of the same assembly (Figure 10.7a and Figure 10.7b). These 3D representations make the wall ties visible within the overall construction system, helping readers better understand how the brick veneer is anchored to the backup wall and how the different layers work together as an integrated system.

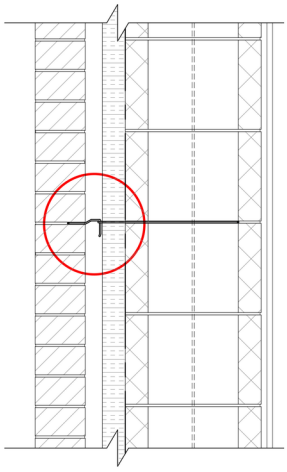
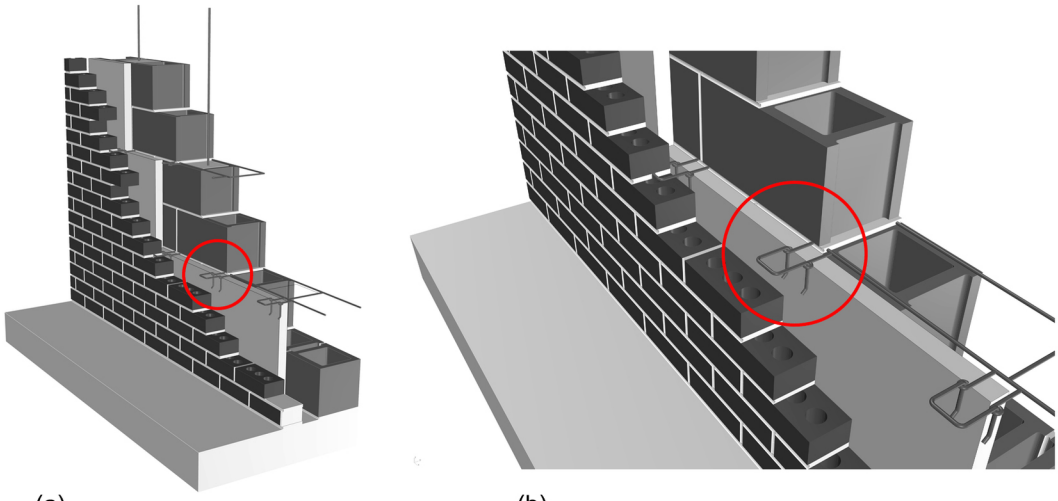


Figure 10.6 Building envelope section with highlighted anchoring system

(Figure 10.6 illustrates the same wall detail presented in Figure 10.3, with the labels removed and the wall tie detail identified by a red circle.)



(a) (b)

Figure 10.7a 3D representation of building envelope detail with highlighted anchoring system; Figure 10.7b Zoomed in 3D image of metal wall tie

(These figures illustrate the three-dimensional model of the wall assembly shown at different scales, highlighting one of the wall ties in the 3D environment to demonstrate its relationship with the corresponding element in the section drawing.)

Advanced visualization and fabrication tools: AR, VR, and 3D printing – The integration of advanced visualization technologies into engineering and architecture education has significant pedagogical implications. While the use of physical three-dimensional models in architectural and engineering education is not new, contemporary digital tools such as augmented reality (AR), virtual reality (VR), and 3D printing expand the possibilities for visualization, interaction, and experiential learning. These tools allow students to engage directly with complex building assemblies in ways that are not possible through conventional two-dimensional drawings alone. In particular, AR and VR can help students develop spatial reasoning skills, improve comprehension of layered wall systems, and better connect construction details with real-world applications. Such technologies also support interdisciplinary collaboration by creating shared visual platforms.

Section 10.7 expands this discussion through a hands-on 3D printing exercise of the envelope detail discussed in this section. The exercise demonstrates how physical fabrication methods can further reinforce students' understanding of construction systems by translating digital models into tangible objects that can be assembled, examined, and discussed in the classroom.

Virtual reality demonstration – To further illustrate the value of VR in providing students and professionals alike with an interactive and examinable representation of a complex three-dimensional object or set of objects, a simple VR demo was developed in support of this text. The demo was created using the same 3D models developed for the 3D printing exercise discussed in Section 10.7; in this case, rather than the models being printed in reality, they are represented digitally and can be manipulated using a VR headset and controllers.

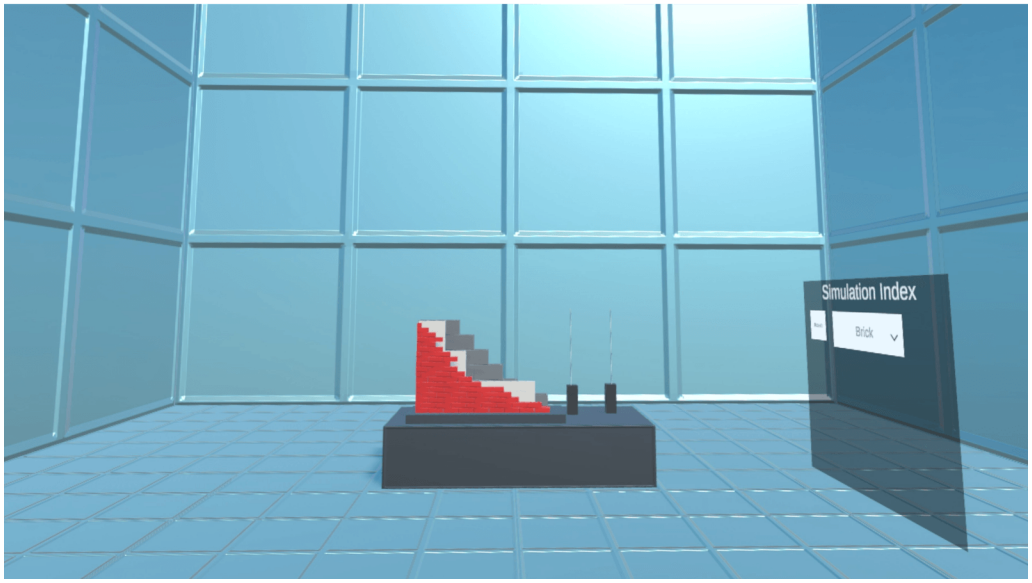


Figure 10.8 Screenshot of the VR demo, showing the wall cutaway model and menu panel

(This figure illustrates the VR demonstration environment, featuring a virtual room that contains a three-dimensional model of the wall assembly.)

This simple demo consists of a small virtual room with a workbench and model set of the parts of a typical CMU/brick wall. Users can "grab" each component and manipulate it; components can be scaled larger or smaller and rotated for inspection. There is also a "Simulation Index" panel that allows the user to select a component and receive additional information about it, including a high-resolution render. When the demo is launched, the model parts are presented in an assembled configuration as they would appear in a real wall. As the player moves them and changes them in size, they may become disorganized. The Simulation Index includes a "reset button" that will return all components to their original sizes and positions.

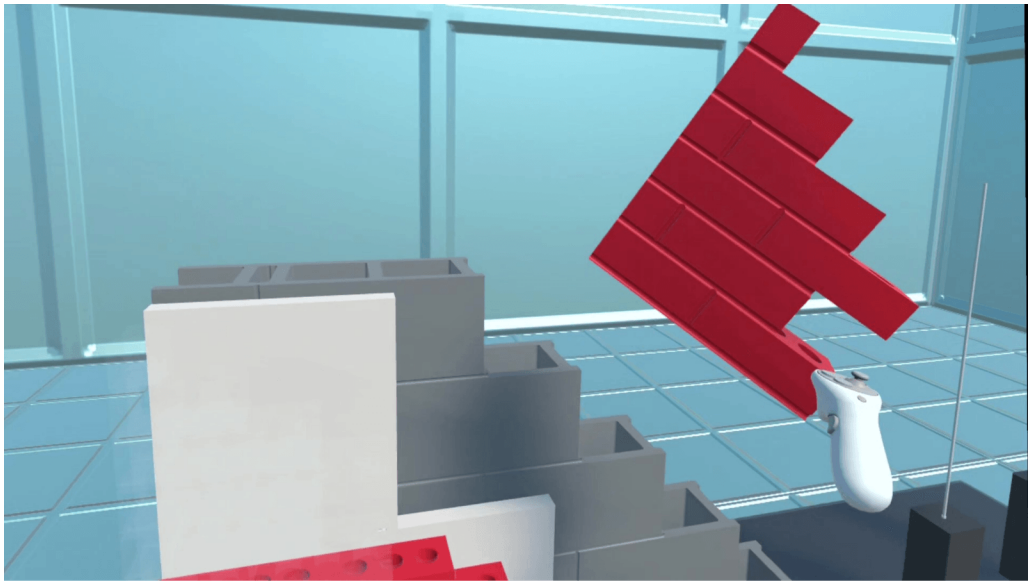


Figure 10.9 Screenshot of the VR demo, showing a user manipulating one of the wall components.

(This figure illustrates a close-up view of the user interacting with the brick cladding through the use of VR equipment.)

This VR demo was developed using the open-source Unity software, and users can "play" it using a Meta Quest VR headset system. The demo has been made available for free on the Meta Store. Readers can obtain a link to the demo by contacting the authors.

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10.7: Integrating 3D Printing for Educational Exercises

Understanding complex structures with 3D visualizations – The preceding section of this text discussed the use of two-dimensional representations in understanding complex engineering and architectural structures, and how 3D visualization can complement them. Orthographic representations are vital for understanding the accurate scale and relationships between design elements, but they can require effort and practice to fully understand how they translate to real, three-dimensional objects.

Interacting with accurate 3D models developed from 2D drawings using Augmented Reality (AR) or Virtual Reality (VR) can aid in understanding an object by allowing the user to interact with it that is more intuitive to the way we perceive the world around us. These models can be built directly from 2D drawings using specialized engineering drawing software; in these cases, the models will be idealized, "perfect" versions of the object. They can also be created from modern 3D scanning techniques such as LiDAR and photogrammetry, discussed earlier in this chapter. These techniques produce a digital 3D representation of an object that more closely reflects it's accurate dimensions in real life, and may incorporate any imperfections or damage, seen or unseen. The accuracy of models produced using this technique will depend on the precision of the imaging equipment and the level of access to the object, among other factors. Whether a 3D model is created from scratch using software or through the use of imaging technology, it is still a virtual representation of an object – a simulation, a construct that require a user to understand a technological interface first before they can actually interact with it.

Exploring models in reality Using 3D printing – A physical model which exists in reality allows a user to manipulate it in a way that is perfectly intuitive because it allows for the use of not only sight but touch in understanding the model, and by extension the object it represents. Creating models from drawings has a long history in architecture and engineering, and most students in these and related disciplines have been asked to produce a model in one form or another for many decades. In years past, these models were handmade using traditional materials and tools. During the last fifty or so years, a new form of manufacturing has been deployed, first in industry and more recently for consumers and hobbyists – 3D printing. Many forms of 3D printing exist, but in all cases, the core theory is that a 3D object can be built from many thin layers stacked together. 3D printing has been entering classrooms and laboratories during the last few decades because it offers a number of advantages over traditional modeling techniques.

- *Cost* – Producing models using traditional methods may require the user to purchase costly raw materials or use specialized tools and methods such as Computer Numerical Control (CNC) machining. 3D printing uses cheap thermoplastic as its raw material, and consumer-grade 3D printers are becoming less expensive while maintaining a high degree of print accuracy.

- *Quick prototyping* – Depending on the size of the model, the type of printer, and the settings used, a 3D printer can create an accurate model in a matter of hours. The model can be immediately inspected, and any necessary alterations can be made in the modeling software to produce a revised version, which itself can be printed relatively quickly. This kind of iterative design process can be greatly accelerated through the use of 3D Printing.

- *Accuracy and reproducibility* – Depending on the manufacturer, 3D printers have a limit to the level of accuracy, detail, and tolerances they can achieve when creating a model. However, because the instructions a 3D printer uses to produce a model come directly from a digital and mathematically-defined representation of the model, they can be used to create many accurate copies of the same model, repeatedly.

- C*ustomized and personalized modeling* – A user can take another person's design and modify it themselves using design software, then print it into reality using a 3D printer. An object designed for one use case can be easily customized for a user's own unique use case. In the context of instruction, this allows texts (such as this one) to include 3D models that a student or instructor can print for themselves to better understand the spatial concepts being taught.

Despite the advantages offered by 3D printing, there are a few cautions and considerations to be aware of when bringing a digital 3D model into the real world using a 3D printer, and in some cases the creation of a useful 3D printed model will require a designer to modify a 3D model before printing.

- *Detail* – 3D printers have specific resolutions, depending on the type and model. Many common consumer-grade 3D printers extrude a thin bead of thermoplastic through a heated nozzle that is 0.4 millimeters in diameter, so that can be thought of as the smallest level of two-dimensional detail a single 3D printed layer can achieve. Other types of 3D printers can achieve much higher detail – Stereolithography (SLA) 3D printing uses liquid resin cured with UV light to build up its layers, so the level of detail is determined by the size of the UV LED pixels used to cure the resin. These pixels are nearly microscopic

nowadays. Since a 3D print is comprised of many stacked layers, the resolution on the vertical Z-axis is determined by the thickness of these layers, and a 3D printed model usually displays "layer line" imperfections on its surface.

- *Tolerances* – In a virtual environment, 3D models may fit together perfectly, since their surfaces are defined by precise mathematical formulae; however, the same models may not fit exactly right once they have been 3D printed. Engineers are familiar with the concept of manufacturing tolerances, whereby the dimensions of a physical object may not precisely match the original design but instead fall within an acceptable range of variation. Some 3D printers can print with tighter tolerances than others with respect to the original model, but there will nearly always need to be an accounting for variation in physical dimensions. This may be due to many factors: thermal expansion of the filament; imperfect representation of fine detail, like tight corners; or the maximum precision of the stepper motors used in the 3D printer. The bottom line is that a 3D model may need to be modified to account for printing tolerances if more than one object is meant to interface together.
- *Mechanical interaction of model components* – In a virtual space, a 3D model that is made up of multiple components, such as those included in this chapter, can simply exist in any given arrangement with respect to one another, ignoring physical realities such as gravity or friction. Real-world, printed models may need to be held together with clips, bolts, rods, or magnets; or, they may need to be supported against gravity to stay aligned with other pieces in the set.
- *Material* – Commercial and consumer-level 3D printing typically use thermoplastic or UV-cured resin to build models one layer at a time. These materials will have vastly different physical properties from the real-world structures they approximate. A component that is made of steel at full scale in real life may deform or be brittle when printed with filament as a model.

3D printing overview – Over the last several decades, 3D printing has become a mature technology that is accessible to consumers, including instructors and students. A deep treatment on how 3D printing works and how to use a 3D printer is out of the scope of this chapter; however, the reader is encouraged to refer to the text ***Introduction to Engineering Drawing and Design*****.** In addition to providing a comprehensive overview of orthographic projections of engineering models and other engineering drawing fundamentals, a focus on 3D printing concludes the text. Module D of Introduction to *Engineering Drawing and Design* introduces students to the fundamentals of 3D printing, including basic principles as well as practical considerations. The Appendix of that text is directed at instructors who wish to run their own 3D printing workshops to support their institutions or classrooms. This section goes into more detail on troubleshooting 3D prints and offering a 3D printing service point to support multiple simultaneous 3D printing projects. In lieu of retreading content already covered by this text, a brief overview of the 3D printing process follows.

The essential principle of 3D printing is that a 3-dimensional object can be created by slicing it into very thin layers, then building each layer up in sequence one at a time. In a digital environment, the original model can be composed of mathematically defined surfaces and vertices in the form of a digital file, usually with the .stl or .obj file type, though there are other file types with varying degrees of propriety to 3D modeling software. These model files must be programmatically sliced into layers to facilitate 3D printing. A very common type of commercial 3D printing is Fused Deposition Manufacturing (FDM), whereby each layer is composed of a thermoplastic that is extruded as a liquid from a heated nozzle and allowed to cool before the next layer is deposited. This requires three degrees of movement – a layer requires the deposition of liquid thermoplastic (called "filament" because it is purchased and used as a long, thin strand of plastic spooled onto a roll) along the X and Y axes, and each layer is built incrementally on the Z axis. 3D printers are composed of a sophisticated arrangement of motors and belts to achieve this, as well as a computer unit to precisely control all movement. Because of the rapid heating and cooling of filament required for this process, the 3D printer also contains an electric heating element and temperature-sensing thermocouple, both of which are also controlled by the printer's computer.

Before a model can be 3D printed, specialized 3D printing "slicing" software must be used to convert the original model into the precise paths that the 3D printer's nozzle must follow to deposit plastic for each layer. The resulting file is called GCODE, which is then loaded onto the 3D printer. The slicing software contributes more than simply breaking a model into layers. To conserve both print time and filament, a 3D model is designed to be mostly empty space inside; the exterior part which is visible is a mere shell (called the "wall"), and a regular geometric pattern fills the interior, called infill. Both are automatically generated by the slicing software, and the user has a great deal of control over these parameters depending on the eventual application of the model. After GCODE is loaded onto a 3D printer via USB or a microSD card, the print can proceed. Often, an FDM 3D print will take several hours. If a print is going to fail, it will however likely fail during the first 20 minutes or so. Again, please refer to the appendix of Introduction to *Engineering Drawing and Design* for tips on troubleshooting 3D prints.

This chapter's 3D models – Included as attachments to this chapter are a collection of .STL 3D model files that were developed to illustrate a commonly deployed brick veneer/CMU wall structure. These model components are intended to fit together in a way that illustrates the complex arrangement of this construction element in reality. They can be inspected as-is using a digital model display application, and there are many such .STL model display programs available. However, these model components were explicitly designed to be 3D printed so a learner can manipulate them and cement their understanding of how the components interface.

These model components were not created from a real structure using a 3D imaging technique like LiDAR or photogrammetry. Rather, they were modeled from scratch using the 3D modeling program Rhinoceros 3D. Development of these 3D models was based on 2D figures published in Allen and Iano's *Fundamentals of Building Construction: Materials and Methods* (2019), specifically referencing figures 10.2 and 10.3 in chapter 10 of that text.

This model is presented for illustrative purposes and is intended to help the reader visualize how a standard wall section is constructed. Collectively, the model's components include a brick façade and a CMU structure underlying it. It includes all the detail that a two-dimensional projection would depict while providing the reader with additional clarity by showing hard-to-visualize details in 3D. For example, the anchoring system between the CMU wall and the brick façade may be easier to understand on this model as opposed to a 2D projection.

These model components were originally drawn using the Imperial system of measurement (feet and inches). When 3D printed, it was designed such that the longest edge of the largest component – the base – is 9 inches (229 millimeters). This represents a 1 foot = 1-½ inches (38 millimeters) scale model to an actual construction feature, or 12.5% scale.

This model includes the following components:

- *Base* – This is not a true architectural feature and is merely a functional component to hold the rest of the model parts together in relation to one another. It includes a series of pegs into which corresponding holes on the other model parts fit.
- *Brick Façade*
- *Foam Insulation Layer*
- *CMU Blocks*
- *Rebar-reinforced Poured Cement* – This component represents a poured bonding element (such as mortar, concrete, etc.) which fills regularly spaced voids in the CMU, and is internally reinforced with rebar. The rebar is modeled to project up from the poured cement in this model for illustrative purposes but would be fully encased in reality. In addition, filled CMU voids are spaced as depicted in this model, and filled voids are offset vertically in actual practice.

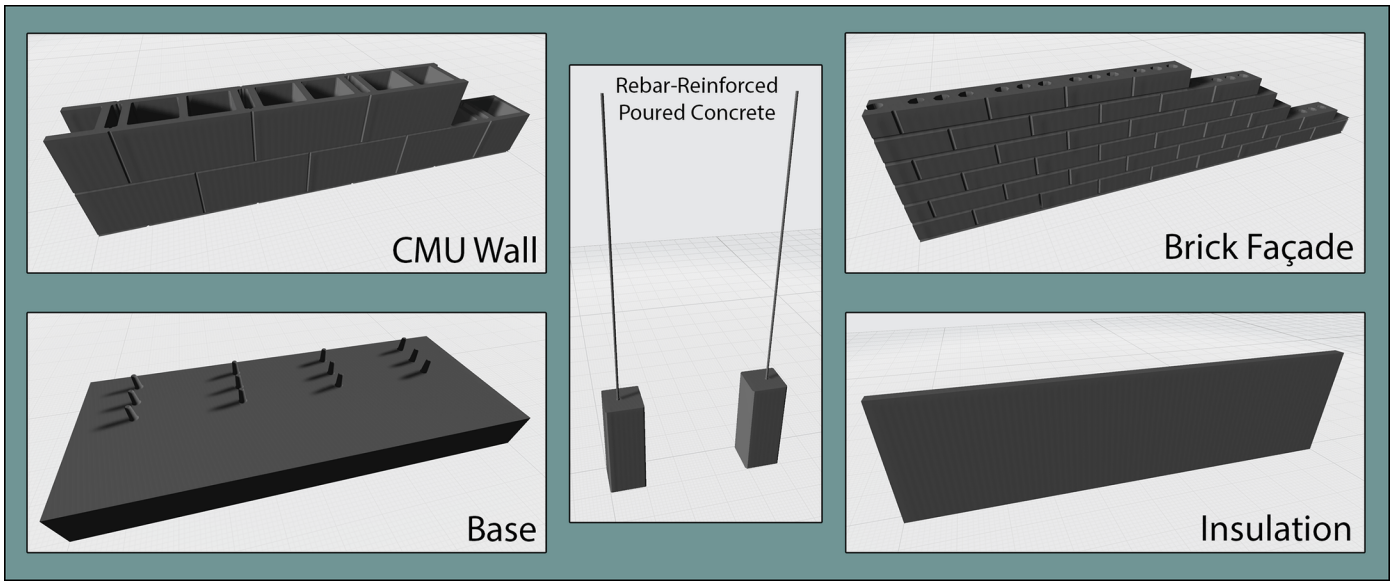


Figure 10.10 Individual components of the CMU/brick façade 3D model set

(This figure shows various components of a brick or CMU wall made using a 3D printer.)

Tips for 3D printing this model set – Once again, if the reader is not familiar with 3D printing fundamentals, they are strongly encouraged to review Module D of *Engineering Drawing and Design*. The following paragraphs will refer to specific 3D printing terminology which can be best contextualized from reading that module.

It should first be noted that this model was designed such that its components are accurately positioned in space with relation to one another, including voids and air gaps. Some sacrifices were made, however, with respect to the level of detail achievable on a typical consumer-grade 3D printer. For example, drip edges and weeps are not included, as well as flashing and vapor-barrier underlayment. These details would be difficult to depict at an accurate scale using 3D printing. The reader could, however,

exercise their own creativity in depicting these features with non-3D printed materials, such as paper, fabric, foil or wire, on their final models.

As previously stated, pegs and holes were added to the components of this model to hold the pieces together. These should fit with only a small amount of play, as print tolerance was taken into account. However, the reader may find it necessary to use a sandpaper to smooth out pegs and prevent sticking.

This model was designed to have two vertical layers of each component to better illustrate its internal structure. The second layer of the brick component includes negative space to accommodate wire anchor clips that hold the brick to the structural CMU wall behind it.

Print scale – The longest edge of the base should be printed at 9 inches (229 millimeters). Smaller scales than this could result in loss of detail or diminished tolerances that can cause components to be unable to slot into the pegs on the base. Larger scales should be possible and potentially even preferable depending on the capabilities of the reader's printer, though the scale factor will of course change according to real-world wall structures. It is important to note that most 3D printing software uses metric measurements by default, so some unit conversion may be required.

Walls and infill – Most slicing programs default to 2 walls with 15% infill, and this default should be sufficient for these models. The reader may choose to increase the wall count of the base to reinforce the pegs that hold components together.

Supports - The models are built from individual (virtual) bricks and CMUs, meaning they have internal voids. For this reason, supports must be enabled during printing, though the supports will be entirely contained within each component model and will not require removal.

Layer height - The default of 0.20 mm layer height should be fine, and users can even experiment with thicker layers to reduce printing time. However, for the poured reinforced concrete components with exposed rebar, thinner layers are preferable due to better layer adhesion on the small cross-section on the rebar specifically; the smaller the better, but 0.10 mm would be recommended.

Print order – The components can be printed in any order, with one recommended exception. Print the two poured reinforced concrete components individually to prevent stringing across the thin rebar elements.

Filament color - The choice of filament colors can be left to the reader's preference, though the included figure depicts one possible palette that approximates real colors. Brick may be printed in a red hue, and insulation can be printed using transparent or translucent filament. CMU may be printed in a light gray to reflect its natural color; the reader may be tempted to do the same for the poured concrete component for this reason, though they may be advised to select a different color to more clearly indicate that it is a separate component from the CMU wall. If the reader chooses to highlight a particular wall feature for a specific reason, they may consider a bright or striking color for that feature.

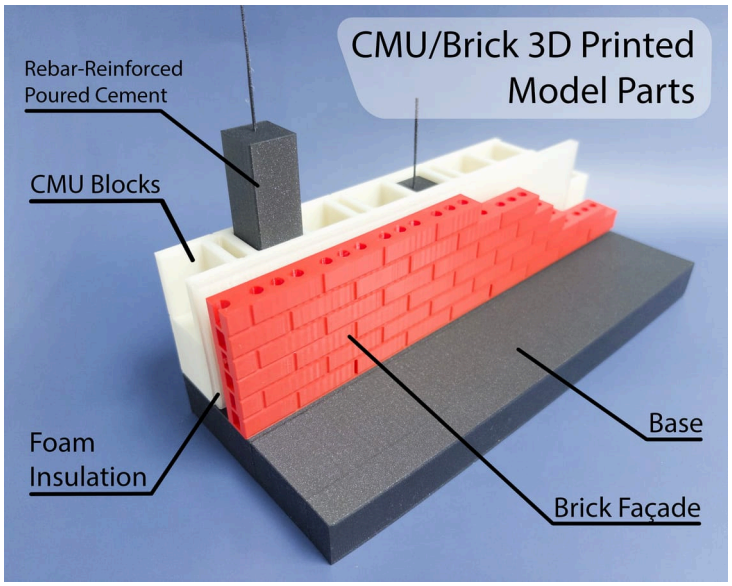


Figure 10.11 The 3D printed CMU/brick façade model

(This figure shows the 3D-printed wall Components assembled together.)

Going beyond this model set – The .STL files included in this chapter are for 3D printing directly and will be of limited use for modifications because they represent only the exterior features of each component. However, this chapter also includes the original Rhino 3D files for each component. These cannot be printed directly without converting them to .STL files, but they include the internal geometries and sub-components of each model which may make modifications easier or more illustrative. For example, in the Rhino 3D files, each brick and CMU is modeled separately, whereas in the .STL versions they are joined into a single solid piece.

The reader may choose to modify the model to represent a variety of conditions and failure modes. One potential application is the incorporation of simulated damage, displacement, deformation, or deterioration of individual components to illustrate how such conditions may manifest in actual structures and how they can be identified during inspection and assessment. Because individual components can be readily replaced, the printed model can also serve as a three-dimensional educational and assessment tool, allowing students and practitioners to evaluate their ability to recognize and diagnose various forms of wall distress, weathering, and material degradation. The inclusion of damaged, corroded, or failed metal anchors and connections may further enhance the instructional value of the model by demonstrating common structural and serviceability deficiencies encountered in practice.

For the model itself, the reader may choose to replace the peg-and-hole construction with something more elevated, like the inclusion of voids into which small magnets can be glued. Such magnets can be obtained easily via online retailers and may provide models with more stability and cohesion. The reader may also experiment with different types of 3D printing filament with different appearances or properties. The models were developed with the assumption that standard polylactic acid (PLA) filament would be used for fabrication. However, thermoplastic polyurethane (TPU) filament may be substituted for selected components, such as the insulation layer, to better simulate the mechanical behavior of actual materials. TPU may also be used for components representing metallic elements to demonstrate deformation mechanisms, including shear and bending, under applied loads. Such material substitutions can enhance the educational value of the model by providing a more realistic representation of the structural response of individual building envelope components.

Advanced and emerging visualization technologies – Significant research activity has evolved in recent years in the integration of AI with VR/AR/MR technologies for automated damage detection and condition assessment in the built environment. Most of these advances have emerged from related application areas such as civil infrastructure inspection (e.g., bridges, tunnels, etc.), construction monitoring, and structural health assessment. In these studies, AI techniques, particularly computer vision and deep learning models, are used to detect, classify, and quantify defects such as cracks, spalling, corrosion, and surface deterioration from images, videos, or point cloud data. VR/AR/MR platforms are subsequently used to visualize detected damage in immersive or semi-immersive environments, enabling enhanced interpretation and decision-making. These AI-driven detection results can be visualized in immersive environments, allowing inspectors and engineers to interact with detected defects in spatial context (Catbas et al., 2022; Davila Delgado et al., 2020; Karaaslan et al., 2019).

Within this research stream, AR-based systems have been developed to overlay AI-detected damage directly onto physical structures during field inspections, while VR and MR environments enable inspectors and engineers to interact with digital representations of assets in remote or collaborative settings. These approaches demonstrate the potential of combining automated damage recognition with immersive visualization to reduce subjectivity and enhance situational awareness during inspections and consistency of inspection results.

Despite the technical maturity of AI algorithms and immersive visualization platforms, their application has largely focused on structural components or generalized building elements. Façade and building envelope systems, characterized by layered assemblies, concealed components, and performance-driven deterioration mechanisms, have received comparatively limited direct attention. As a result, current AI-integrated VR/AR/MR frameworks often do not explicitly address building envelope-specific damage types such as moisture intrusion, delamination behind cladding, sealant failure, thermal performance degradation, etc. This gap highlights a clear need for future research to adapt and extend existing methodologies to address the unique inspection and management needs of building envelopes.

Another notable research and development trend is the emergence of low-cost VR, AR, and MR hardware and software platforms, which has significantly lowered the barrier to adoption of immersive technologies across many engineering disciplines. Head-mounted displays, mobile-device-based AR applications, and standalone VR systems now provide sufficient performance for visualization, interaction, and spatial tracking at a fraction of the cost of earlier professional systems. In parallel, open-source and commercial software tools for 3D reconstruction, photogrammetry, mobile LiDAR, and real-time rendering have become

increasingly accessible to researchers and engineers.

These developments have enabled broader experimentation with immersive inspection workflows in both academic and applied research settings. In particular, mobile AR solutions integrated with smartphone cameras and tablets have shown promise for on-site visualization and data capture, while desktop and standalone VR systems support remote inspection and collaborative review of inspection data. For building envelope management, such tools present a practical pathway for integrating AI-assisted analysis and immersive visualization into routine inspection and maintenance practices without requiring specialized equipment.

Nevertheless, despite improved accessibility, research focusing on the systematic application of immersive technologies to façade inspection and lifecycle management remains limited. Most existing studies emphasize technical feasibility rather than long-term deployment, data integration, or decision-support capabilities tailored specifically to façade systems. This gap underscores the need for façade-focused frameworks that leverage affordable VR/AR/MR technologies while addressing real-world inspection constraints and performance objectives. The integration of affordable immersive tools with AI-driven damage detection presents an opportunity to widely use advanced façade assessment techniques, enabling more frequent inspections, improved documentation, and broader adoption across building portfolios.

The convergence of artificial intelligence (AI), advanced visualization technologies, and digital twin concepts indicates a future in which building envelope inspection and management can be performed in a more remote, intelligent, and data-driven manner. For building envelopes – particularly those that are tall, geometrically complex, or difficult to access – remote, AI-assisted inspection methods offer substantial advantages in terms of safety, operational efficiency, and data reliability. When integrated with AI-driven analytics, these platforms enable a shift in façade management practices from predominantly reactive inspection toward predictive and proactive maintenance strategies.

Despite these advantages, façade-specific implementations of remote and AI-assisted inspection remain largely conceptual or exploratory. While related methods have been developed for bridge systems and general building components, building envelope–focused approaches must be specifically tailored to address the performance requirements, dominant failure mechanisms, and regulatory constraints unique to façade systems. The development of AI-assisted visualization and analysis systems that explicitly support façade diagnostics, damage progression assessment, and repair prioritization would represent a significant advancement in building envelope management practice.

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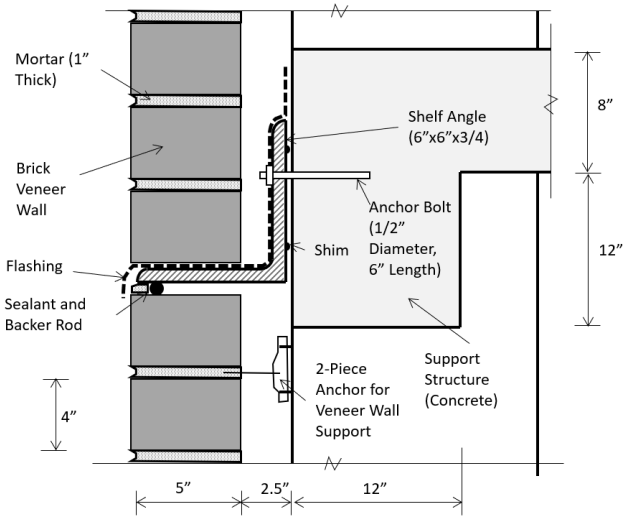
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10.9: Exercises

E10.1 This exercise is intended for the creation of a 3D model from a 2D image. The accompanying sketch demonstrates a vertical cross-section of a brick wall with support anchors and flashing.

a) Prepare a 3D sketch showing the key features of the damage shown in the 2D.

b) Highlight potential hidden damage areas (e.g., missing anchor to support veneer wall, lack of flashing, deformed shelf angle).



Note: Dimensions are in inches and shown not to scale

(This figure shows the cross-section of a brick wall resting on a shelf angle. An anchor to prevent wall movement, and flashing are shown. The detail of the attachment of the shelf angle to the concrete structure via anchors is also shown.)

E10.2 Augmented reality façade inspection simulation.

Objective: Familiarize students with the concept of AR overlay during an inspection.

Task: Use a simple AR app (or any available AR viewer).

a) Prepare a 3D sketch showing the key features of the damage shown in the 2D.

b) Highlight hidden damage areas.

E10.3 3D Printing a façade component.

Objective: Understand how façade units attach to structural systems.

Task: Convert a basic 3D model (provided or created) into a printable file (.stl format).

a) Print a small replica of a façade segment (e.g., brick veneer over backup wall, terra cotta cladding).

b) Identify attachment points, joints, and hidden support features on the model.

E10.4 Damage assessment and repair planning scenario.

Objective: Plan a repair based on observed and inferred damage.

Task: Using a provided 3D model (physical or digital), identify visible and hidden defects.

a) Prepare a short-written report outlining possible causes and proposed repair steps.

b) Mark repair areas using AR or color-coding in the 3D model.

E10.5 Comparative visualization exercise.

Objective: Learn how visualization changes damage perception.

Task: View a façade through traditional 2D photos and through a 3D VR environment.

a) Note differences in how much damage is detected and understood in each format.

b) Write a brief reflection on the advantages of 3D/VR visualization.

E10.6 Create a simple VR walkthrough.

Objective: Create an immersive inspection experience.

Task: Use a free or basic VR tool.

a) Develop a simple VR walkthrough of a façade showing both good and damaged areas.

b) Embed clickable “notes” in VR highlighting hidden or subtle damages.

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11: APPENDIX A - ILLUSTRATIVE EXAMPLES

Appendix Synopsis

This appendix offers illustrative examples as additional materials to various chapters of the book. Specific references to the sections in chapters pertinent to individual examples are provided.

11.1: Illustrative Examples for Chapter 1

11.2: Illustrative Examples for Chapter 2

11.3: Illustrative Examples for Chapter 3

11.4: Illustrative Examples for Chapter 4

11.5: Illustrative Examples for Chapter 5

11.6: Illustrative Examples for Chapter 6

11.7: Illustrative Examples for Chapter 7

11.8: Illustrative Examples for Chapter 8

11.9: Illustrative Examples for Chapter 9

11.10: Illustrative Examples for Chapter 10

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11.1: Illustrative Examples for Chapter 1

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11.2: Illustrative Examples for Chapter 2

Example 2.1 (Refer to Section 2.2)

A veneer wall in a single-story building of 15 feet (4.57 meters) height has 4 inches (102 millimeters) of nominal thickness and is resting on a footing of plain concrete. Compute the dead weight of the wall applied on the footing. Also check to find out whether the stresses on the mortar and on the concrete in the foundation are within the respective strength values. Use the LRFD method and a weight per unit area of 42 pounds per square foot (2 kilonewtons per square meter) for a 4-inch (102-millimeter) brick veneer wall.

Solution:

For a 4-inch (102-millimeter) brick veneer wall, the weight W is

W =15x42 = 630 pounds per foot (9.19 kilonewtons per meter)

For a 4-inch (102-millimeter) wall, the effective thickness T_E is 4.0 minus 0.375 or 3.625 inches (92 millimeters). Since this value is for one foot (or one meter) of the length of the footing, the compressive stress on the footing because of the weight of the wall is

$f_c = \frac{630}{3.625 \times 12}$ = 14.5 pounds per square inch (0.10 megapascals)

This stress is from the dead weight of the wall, thus the factored applied stress (using a load factor equal to 1.4) will be 1.4x14.5 =20.3 pounds per square inch (0.14 megapascals). Using the Load and Resistance Factor Design (LRFD) method, the factored compressive strength of concrete and Type N mortar are respectively (applying 0.65 capacity reduction factor):

Concrete compressive capacity = 0.65x$(0.85f'_c)$ = 0.65x(0.85x3,000) = 1,658 pounds per square inch (11.43 megapascals)

Mortar compressive capacity = 0.65x750 = 488 pounds per square inch (3.36 megapascals).

Thus, the applied stress is within the strength values for both the mortar and the concrete and the wall thickness is acceptable.

Example 2.2 (Refer to Section 2.3)

For the veneer wall given in Example 2.1, determine the maximum allowable value for the spacing between anchors.

Solution:

The effective wall thickness for the computation of the section modulus is 3.625 inches (92 millimeters), as computed in the solution for Example 2.1. Furthermore, the wall density w in pounds per inch is 42 divided by 144 (the area of one square foot). This results in 0.292 pounds per square inch (2.01 kilonewtons per square meter). Using the equation given in Section 2.3 (Chapter 2) for the S, the maximum value for anchor spacing is

$S = \sqrt{\frac{8 \times 3.625 \times (15 \times 12) \times 15}{6 \times 0.292}}$ = 211.4 inches = 17.6 feet (5.4 meters).

Example 2.3 (Refer to Section 2.3)

In Example 2.2, if an edge distance equal to 1 foot (0.3 meters) is used in the angle, determine the maximum edge distance that can safely be used.

Solution:

Using the equation given in Section 2.3 for S_E, and substituting proper values for the parameters T_E, w, F_b and h, the value of S_E is computed to be equal to 8.8 feet (2.7 meters). Certainly, this value is larger than the available spacing, which is 1 foot (0.3 meters). Thus, the design is acceptable.

Example 2.4 (Refer to Section 2.3)

An angle of the size 6-inch by 6-inch and 3 quarters of an inch in thickness is used as a shelf angle to support a 4-inch (102-millimeter) veneer wall of 13 feet and 4 inches (4.1 meters) in height. The cavity gap used is 2 inches (51 millimeters). The spacing for the anchor is taken as S equal to 4 feet (1.22 meters).

Determine whether the spacing selected is adequate for the veneer to support itself.

Solution:

The effective wall thickness for the computation of the section modulus is 3.625 inches (92 millimeters). Furthermore, the wall weight density is 0.292 pounds per square inch (2.01 kilonewtons per square meter) and h is 13 feet and 4 inches or 160 inches total (4.1 meters). Using the Equation given in Section 2.3 for S, the maximum value for anchor spacing is

$S = \sqrt{\frac{8 \times 3.625 \times (160) \times 15}{6 \times 0.292}}$ = 199.3 inches = 16.6 feet (5.1 meters), which is larger than the available spacing of 4 feet (1.22 meters). Thus the spacing selected is acceptable.

Example 2.5 (Refer to Section 2.3)

In an application, the angle shown in Figure E2.5 is used as a shelf angle to support a 4-inch veneer wall of 9.5 feet (2.9 meters) in height. The cavity gap used is 1 inch (25 millimeters). The spacing used for the anchor S is 1 foot (0.3 meters).

Determine whether the spacing selected is adequate for the veneer to support itself.

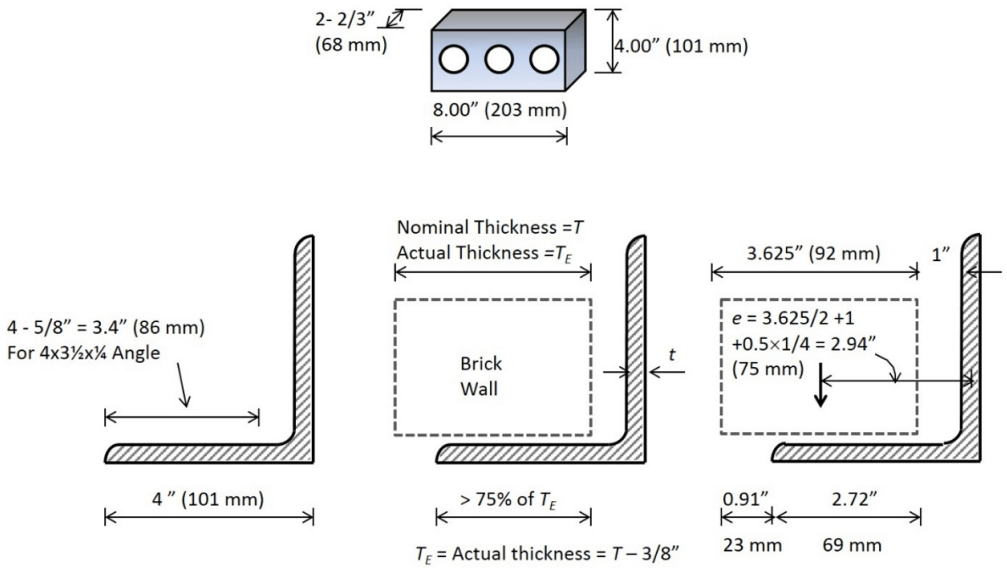


Figure E2.5 Shelf angle for Example 2.5

(This figure shows the relative position of a 4-inch veneer wall on the leg of an angle. The illustrations demonstrate the distance of the load, which is from the weight of the wall, to the center of the vertical leg of the angle.)

Solution:

Again, the effective wall thickness for the computation of the section modulus is 3.625 inches (92 millimeters). Furthermore, the wall weight density is 0.292 pounds per square inch (2.01 kilonewtons per square meter) and h is 9.5 feet or 114 inches (2.9 meters). Using the equation given in Section 2.3 for S, the maximum value for anchor spacing is

$S = \sqrt{\frac{8 \times 3.625 \times (114) \times 15}{6 \times 0.292}}$ = 168.2 inches = 14.0 feet (4.3 meters), which is larger than the available spacing of 1 foot (0.3 meters). Thus the spacing selected is acceptable.

Example 2.6 (Refer to Section 2.6)

a) Compute the linear expansion for a 25-feet (7.6-meter) wall of brick masonry and for 150 degrees Farenheit (83 degrees Celsius) temperature change.

b) If we would like to maintain a quarter of an inch (6 millimeters) width for a joint, determine the maximum distance between any two joints.

Solutions:

a) From Table 2.3 of Chapter 2, the coefficient of thermal expansion α for a brick wall is 3.1 times 10 to the power of -6 per degree Farenheit (5.6 times 10 to the power of -6 per degree Celsius). Thus using the equations given in Section 2.6

$\Delta_L = \alpha L(\Delta T)$ = 0.0000031×(25×12)×150 = 0.14 inches (3.5 millimeters)

b) Using 0.25 inches (6 millimeters) as the value for b_j, and from equations given in Section 2.6, the limits for the distance L between joints will be

$L \leq \frac{b_j}{\alpha(\Delta T)}$ = (0.25)/(0.0000031×150) = 537.6 inches = 44.8 feet (13.7 meters) for distance between two adjacent joints. One half of this value, which is 22.4 feet (6.8 meter), is used for the distance between the wall corner and the first joint.

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11.3: Illustrative Examples for Chapter 3

Example 3.1 (Refer to Section 3.3)

In a one-story building, an 8-inch masonry wall supports the load from the roof. The total factored load transferred from the roof to one foot of the length of the wall R_u is 520 pounds per foot (7.6 kilonewtons per meter). Note: the DL and snow determined this load, and the load factor used for DL is 1.2. The height of the wall is 18 feet (5.5 meters). A ledger beam 2-inch by 12-inch (51 by 305 millimeters) is used to attach the floor joists to the wall. Note: reinforced CMU masonry has a density of about 125 pounds per cubic foot (2,002 kilograms per cubic meter).

a) Determine the moment developed because of the reaction force acting on one side of the wall.

b) Compute the eccentricity for the resultant of compression forces and decide whether there will be any tension developed in the wall cross section.

Solution:

a) Based on the above information, and knowing that a 2-inch by 12-inch beam has a width of 1.5 inches (38.1 millimeters), the eccentricity e of the load from the roof is equal to one half of the width of the wall plus width of the ledger beam. Thus

e = 8.0/2 +1.5 = 5.5 inches (139.7 millimeters)

The external moment M_u is

$M_u = R_u \times e$ = 520x5.5 = 2,860 inch.pounds (0.32 meter.kilonewtons)

b) The weight of the wall for one foot of the length (considering 1.2 load factor) is

W_u = 1.2x18x(8/12)x125 = 1,800 pounds per foot (26.3 kilonewtons per meter)

Thus, the total factored axial load is 1,800+520 = 2,320 pounds per foot (33.9 kilonewtons per meter). Thus the total eccentricity e' of the applied load on the masonry considering the effect of the moment is

e' = 2,860/2,320 = 1.23 inches (31.2 millimeters). This is smaller than one sixth of the wall thickness (8 inches divided by 6), which is 1.33 inches (33.8 millimeters). Therefore, no tension will develop in the wall cross section.

Example 3.2 (refer to Section 3.4.1)

A roof system which is made up of joists spanning 30 feet (9.1 meters) each and spaced at 2 feet (0.6 meters) is supported by a concrete wall of 14.0 feet in height (4.3 meters) and 8 inches (203 millimeters) in thickness. The roof is used as a recreational deck; and as such, it has a design live load (LL) of 100 pounds per square foot (4.8 kilonewtons per square meter) per City of Chicago. The dead load (DL) including the joist weights and finishing is 25 pounds per square foot (1.2 kilonewtons per square meter). The governing load combination will be 1.2 for the dead load plus 1.6 times the live load. The tributary width for the load on each joist is 2.0 feet (0.61 meters).

a) Compute the factored applied load P_u over the effective length of the wall (including the self-weight of the wall).

b) Compute the nominal resistance P_n and decide whether the wall design is acceptable.

Solution:

a) The factored uniform load q_u on each joist is

q_u = 2.0x(1.2x25+1.6x100) = 380 pounds per foot (5.6 kilonewtons per meter)

and the reaction R_u is

$R_u = q_u L/2$ = 380x30/2 = 5,700 pounds (25.4 kilonewtons)

Considering a bearing width of 4 inches (102 millimeters) for each joist and a wall thickness of 8 inches (203 millimeters), the effective length of this wall for stress computation will be the smaller of 24 inches and 4.0 plus 4 times 8.0 inches (which is 36 inches or 914 millimeters). Thus the effective length is 24 inches. To compute the factored compressive load, we also need to compute the weight of the wall. The weight over the effective length of 24 inches will be (considering 150 pounds per cubic foot concrete density and 8 inches of thickness or 23.6 kilonewtons per cubic meter).

Wall self-weight (with load factor) = 1.2x2.0x14.0x150x(8/12) = 3,360 pounds (14.9 kilonewtons)

Thus, the factored compressive load is

P_u = 5,700+3,360 = 9,060 pounds (40.3 kilonewtons)

b) We use the equation given in Section 3.4 of Chapter 3 for computing the nominal axial load capacity of the wall. Parameters used are explained below.

k is taken as 1

h is equal to 8 inches

ℓ is equal to 14 feet or 168 inches

Gross cross-section area over the effective length, A_g, is 24 times 8 or 192 square inches

Concrete compressive strength, f'_c, is equal to 4,000 pounds per square inch

Thus

$P_n = 0.55 \times 4,000 \times 192[1 - (\frac{1.0 \times 168}{32 \times 8})^2]$ = 240,488 pounds (1,069.7 kilonewtons)

The resistance reduction factor ϕ is 0.65 for a compression member. Thus, the factored resistance of the wall is

ϕP_n = 0.65x(240,488) = 156,317.2 pounds (695.3 kilonewtons), which is certainly larger than the factored total applied load. Therefore, the wall thickness is acceptable.

Example 3.3 (Refer to Sections 3.4.1 and 3.4.2)

Figure E3.3a shows an isometric view of a one-story building used as a restaurant. The south and north walls are each 11-inch thick reinforced concrete load bearing walls carrying the entire roof load. The roof is used as an open seating area with a dead load of 30 pounds per square foot (1.4 kilonewtons per square meter) and a live load of 100 pounds per square foot (4.8 kilonewtons per square meter). The snow load is estimated as 22 pounds per square foot (1.1 kilonewtons per square meter). The height of the walls each is 18 feet (5.5 meters) measured from the footing to the top of the parapet wall with 5 feet (1.5 meters) of the wall below grade. The height of the parapet wall is 3.5 feet (1.1 meters). Furthermore, north and south walls are working as shear walls against an east-west wind pressure of 20 pounds per square foot (1.0 kilonewtons per square meter). This wind pressure includes all necessary factors including the effect of the negative pressure that occurs in the leeward side. The resistance of concrete f'_c is 3,000 psi pounds per square inch (20.7 megapascals)

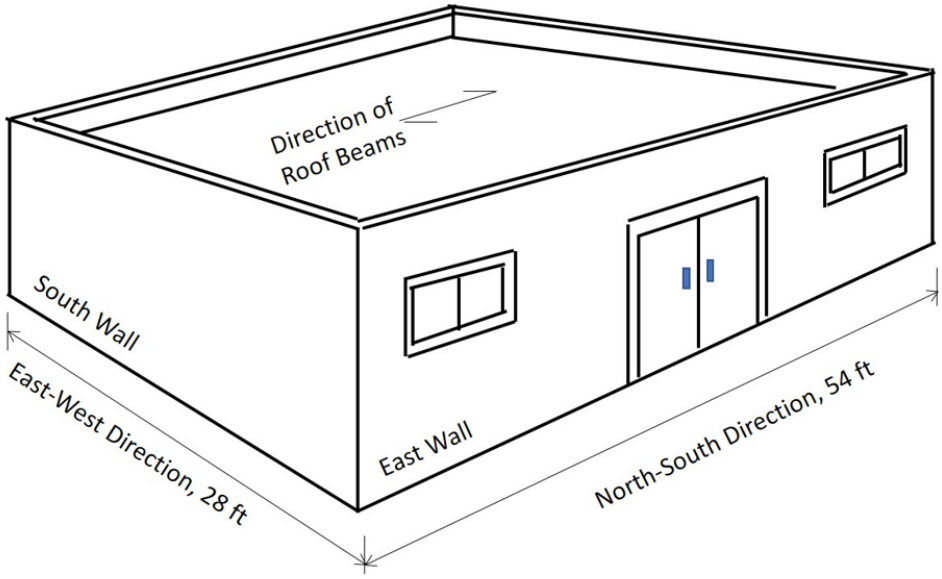


Figure E3.3a Schematics of building in Example 3.3

(This figure shows an isometric view of a one-story building with 54 feet of length in north-south and 28 feet in east-west direction).

a) Determine the factored axial and in-plane shear loads considering the most critical load combination according to ACI for the north and south walls.

b) Using the simplified method, determine the factored axial load capacity and decide whether the wall thickness selected is acceptable.

c) Design reinforcement for the walls using number 4 reinforcing bars.

Solution:

a) We first compute the axial and shear forces on the walls individually for the dead, live, snow and wind load. Then using the ACI load factors, we determine which load factor combinations will govern the wall design. The ACI load factors are given in the supplements to Chapter 3. However, in this example, only four loads, i.e., dead load (D), roof live load (Lr), snow (S) and wind load (W) are involved. Thus, the pertinent load combinations are as follows

1.4D

1.2D+1.6Lr+0.8W

1.2D+1.6W+1.0Lr

0.9D+1.6W

Notice that the load combinations indicate Lr or S; and since the roof snow load (S) is smaller than the roof live load (Lr), it does not appear in any of the above load combinations.

The tributary width for gravity loads on each wall is 54.0 divided by 2 or 27.0 feet (8.2 meters). We assume the roof beams (spanning over 54 feet or 16.5 meters) are close enough such that the roof loads are linear load throughout the length of the north and south walls.

Axial load due to the roof dead load is therefore 27.0 multiplied by 30, which is 810 pounds per foot (11.8 kilonewtons per meter)

Self-weight of each wall is 18 times 11-12th of an inch times 150 or 2,475 pounds per foot (36.1 kilonewtons per meter)

Total dead load: P_D is equal to 810+2,475 or 3,285 pounds per foot (47.9 kilonewtons per meter)

Axial load from roof live load: P_{Lr} is equal to 27 multiply by 100 or 2,700 pounds per foot (39.4 kilonewtons per meter)

The computation of the in-plane shear in north and south walls requires information about the way the roof framing system is designed. The wind in the east-west direction will create a force equal to one half of the total pressure at the roof level. The roof framing will need to be properly designed with horizontal bracing (lateral and diagonal) to act as a diaphragm so that it can transfer this force to the north and south walls in the form of the shear at the top (see Figure E3.3b). Thus, using an approximate computation, the in-plane shear in north and south walls each will be a quarter of the total wind pressure in the east-west direction. Considering the dimension of the building and the height of the wall above ground (which is 18 minus 5 or 13 feet), the total force from the wind (say blowing on the east wall) will be

13 times 54 times 20 or 14,040 pounds (62.5 kilonewtons)

Therefore, the in-plane shear V_W from wind in north and south wall each is equal to 14,040 divided by 4 or 3,510 pounds (15.6 kilonewtons).

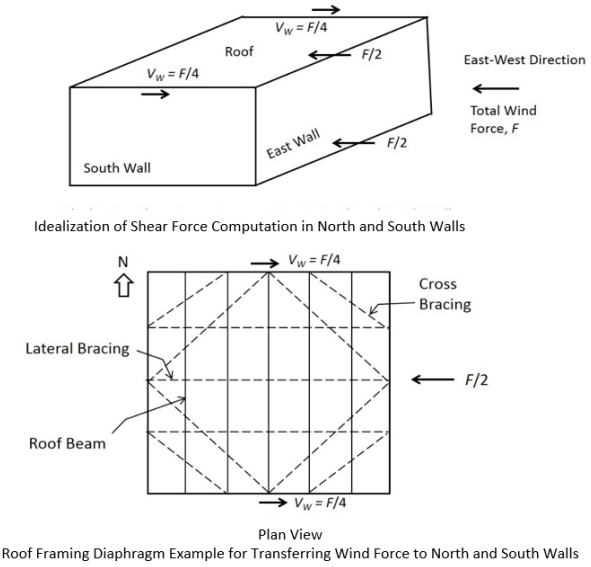


Figure E3.3b Shear forces in north and south walls from wind in east-west direction

(This figure shows the distribution of the wind force to the walls resisting it. An image of the roof bracing is also shown indicating how the wind force transmitted to the roof is distributed to walls parallel to the wind direction).

The shear force also creates compression and tension axial loads in the wall because of the moment it creates. Since the shear force acts at the roof level; and the height of the parapet wall is 3.5 feet (1.1 meters), thus the total moment at the base of the wall is

M = 3,510x(18 – 3.5) = 50,895 foot-pounds (69.0 meter-kilonewtons)

The distribution of stress on the wall because of this moment is triangular as shown in Figure E3.3c.

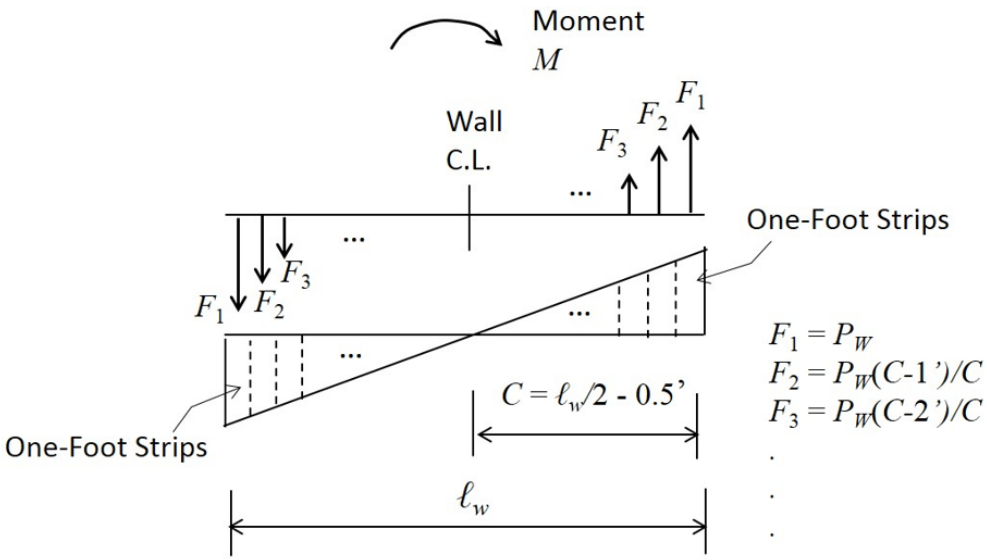


Figure E3.3c Idealization of axial forces developed in the wall as a result of moment *M* which is produced by the in-plane shear force

(In the idealization shown in this figure, the wall is divided into one-foot strips with axial forces $F_1, F_2, F_3,$ etc.)

Using the discrete system of forces (shown in Figure E3.3c) the value of the maximum axial force F_1 is equal to P_W (which is the force in one foot of the width of wall at the extreme ends) can easily be computed by equilibrium of moment M with the moments of forces $F_1, F_2, F_3,$ etc. These forces are related to the force P_W as shown in Figure E3.3.c. Using the equilibrium

$$M = 2[F_1C + F_2(C-1.0) + F_3(C-2.0) + ...] = 2[C + (C-1.0)(C-1.0)/C + (C-2.0)(C-2.0)/C + ...]P_W$$

Distance C in this example is 28 divided by 2 minus 0.5, which is 13.5 feet (4.1 meters). Thus, using the equilibrium

$$M = 2[13.5F_1 + 12.5F_2 + 11.5F_3 + ...] = 2[13.5 + 12.5(12.5/13.5) + 11.5(11.5/13.5) + ...]P_W$$

or

50,895 = 135.33P_W and thus

P_W is equal to (+) or (-) 376.1 pounds per foot (5.5 kilonewtons per meter)

The plus sign indicates compression; while the negative sign means tension. An alternative approximate and conservative method to compute the maximum axial load is to consider one foot of the wall at each end (as seen in Fig. 3.3c, these are locations in the wall that are under the largest axial load F_1 which is equal to P_W and then to use the stress on the wall cross section to compute P_W. Considering this approximate method, the maximum stress on the wall because of moment M is

Compression or tension stress on cross section of the wall =M/S, where S is the section modulus, which is equal to $h\ell_W^2/6$.

And the maximum axial load is

P_W = (+) or (-) $h \times 1.0 \times (M/S)$

and substituting for S

P_W = (+) or (-) $6M/\ell_W^2$ = (+) or (-) 389.5 pounds per foot (5.7 kilonewtons per meter)

The following table summarizes the results for the four load combinations using the ACI load factors in computing the factored axial and shear forces.

Table E3.3 Factored axial load and in-plane shear

Load Combination	Axial Load P_u (pound per foot)	Shear Load V_u (pounds)
1.4D	1.4x3,285 = 4,599	N/A
1.2D+1.6Lr+0.8W	1.2x3,285+1.6x2,700+0.8X(±389.5) = 8,573.6 or 7,950	1.2x0+1.6x0+0.8x3,510 = 2,808
1.2D+1.6W+1.0Lr	1.2x3,285+1.6x(±389.5) +1.0x2,700 = 7,265.5 or 6,018.8	1.2x0+1.6x3,510+1.0x0 = 5,616
0.9D + 1.6W	0.9x3,285+1.6x(±389.5) = 3,579.7 or 2,333.3	0.9x0+1.6x3,510 = 5,616

From Table E3.3, the following values are then suggested as the maximum factored axial and shear loads.

P_u = 8,753.6 poubds per foot (125.1 kilonewtons per meter)

V_u = 5,616 pounds (25.0 kilonewtons per meter)

(b) In most cases, the roof beams are designed with adequate support area where the axial load eccentricity e is practically zero (less than 1-6th of the wall thickness), and as such, the simplified equation for P_n can be used.

$$P_n = 0.55 f'_c A_g \left[1 - \left(\frac{k\ell_c}{32h}\right)^2\right]$$

Since, we are considering only one foot of the length of the wall

A_g =12xh = 12x11 = 132 square inches (85,161 square millimeters)

k =1

ℓ_c = 18x12 = 216 inches (5.5 meters)

$P_n = 0.55 \times 3,000 \times 132 \left[1 - \left(\frac{1.0 \times 216}{32 \times 11.0}\right)^2\right]$ = 135,787.5 pounds per foot (1,981.7 kilonewtons per meter)

Using a resistance reduction factor of ϕ equal to 0.65

ϕP_n = 0.65x135,787.5 = 88,261.9 pounds per meter (1,288.1 kilonewtons per meter), which is larger than P_n of 8,573.6 pounds per foot (125.1 kilonewtons per meter), and wall thickness is acceptable.

(c) For this part, we must first compute the shear resistance V_c of the wall.

$$V_c = 2\lambda\sqrt{f'_c}hd$$

Where d is equal to 0.8 times ℓ_W which is 0.8x(28x12) = 268.8 inches (6.8 meters), and

$V_c = 2 \times 1 \times \sqrt{3,000} \times 11 \times 268.8$ = 323,901.2 pounds (1,440.8 kilonewtons)

Since $V_u = 5,616 < 0.5\phi V_c = 0.5 \times 0.75 \times 323,901.2 = 121,463.0$ pounds (540.3 kilonewtons), we use the steel ratios recommended in Table 3.3 of Section 3.4 of Chapter 3.

Thus

Vertical steel cross-section area in square inch per foot: $A_{sv} = \rho_t bh$ = 0.0012x12.0x11.0 = 0.16 square inches per foot (340 square millimeters per meter)

horizontal steel cross-section area in square inch per foot: $A_{sh} = \rho_t bh$ = 0.0020x12.0x11.0 = 0.26 square inches per foot (550 square millimeters per meter)

Since we use two curtains of steel bars, one half of each steel will be in front face, the other in the back face. Also, because 5 feet of the wall will be in the ground, we use 3.0 inches (76 millimeters) of concrete cover for steel protection.

Vertical bars = 0.16/2 = 0.08 square inches per foot (170 square millimeters per meter). Using #4 bars with an area equal to 0.2 square inches (129 square millimeters), the spacing will be (0.2/0.08)x12 = 30 inches (762 millimeters). However, this exceeds one of the limits, which is 18 inches. Thus,

Use #4 at 18 inches (457 millimeters) on centers for vertical bars on each face.

Horizontal bars = 0.26/2 = 0.13 square inches per foot (257 square millimeters per meter). Using #4 bars, the spacing will be (0.2/0.13)x12 = 18.5 inches (470 millimeters). However, this exceeds one of the limits, which is 18 inches. Thus,

Use #4 @ 18 inches (457 millimeters) on centers for horizontal bars on each face.

The details of the reinforcement are shown in Figure E3.3d.

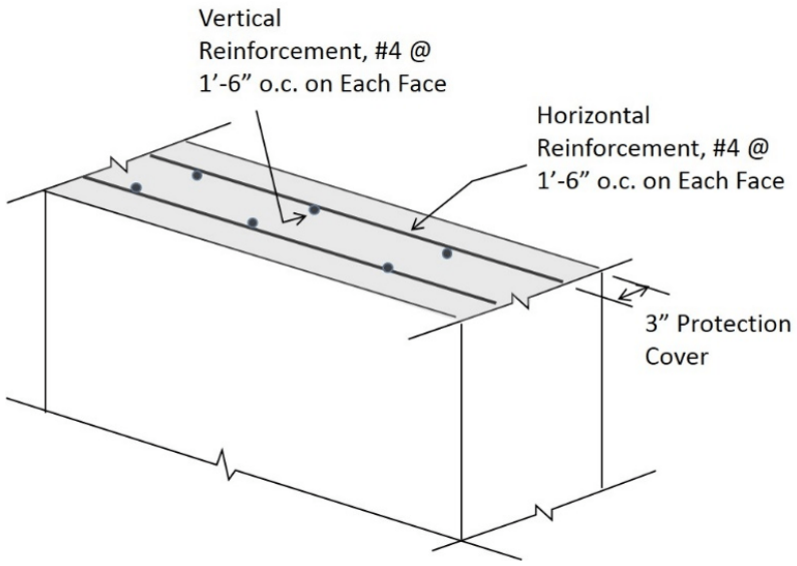


Figure E3.3d Details of reinforcement for Example 3.3

(This figure shows the position of the reinforcing bars in the wall.)

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11.4: Illustrative Examples for Chapter 4

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11.5: Illustrative Examples for Chapter 5

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11.6: Illustrative Examples for Chapter 6

Example 6.1 (Refer to Section 6.2)

A masonry parapet wall has a height of 3 feet (0.9 meters). Its effective thickness is 11.8 inches (300 millimeters). Compute the stresses developed in the mortar for a wind load of 25 pounds per square foot (1.2 kilonewtons per square meter). The wall density is 125 pounds per cubic foot (2,002 kilograms per cubic meter).

Solution:

The bending moment from wind when considering the wall as a cantilever is

$M = \frac{qL^2}{2}$

Where q is wind load (25 pounds per linear foot, or 0.4 kilonewtons per meter) considering one foot of length of the wall). Thus

$M = \frac{25 \times (3.0^2)}{2}$ = 112.5 foot.pounds (0.15 meter.kilonewtons)

The section modulus and the cross-sectional areas for one-foot of the cross section, respectively, are

$S_x = \frac{bd^2}{6} = \frac{12 \times (11.8)^2}{6}$ = 278.5 cubic inches (4,563,797 cubic millimeters)

$A = 12 \times 11.8$ = 141.6 square inches (91,355 square millimeters)

Thus, the bending stress from wind pressure, tension in windward and compression in leeward, is

$f_w = \frac{112.5 \times 12}{278.5}$ = 4.8 pounds per square inch (0.033 megapascals)

The compression stress from self-weight is

$f_c = \frac{3.0 \times 125}{141.6}$ = 2.6 pounds per square inch (0.018 megapascals)

Thus, the total stress is computed as follows

Windward stress = 4.8 – 2.6 = 2.2 pounds per square inch or 0.015 megapascals (tension)

Leeward stress = 4.8 + 2.6 = 7.4 pounds per square inch or 0.051 megapascals (compression)

The tensile resistance of most engineered mortars is about 15 pounds per square inch (0.103 megapascals) as the lowest value. Therefore, considering a safety factor of 1.6, the allowable tensile stress will be about 15/1.6 = 9.4 pounds per square inch (0.065 megapascals). Thus, the wall will be ok under the wind pressure.

Example 6.2 (Refer to Section 6.3)

A flat roof has the configuration shown in the Figure E6.2a. The height of the obstruction ($h_r = h_0$ is equal to 7 feet (or 2.1 meters). The length of upper roof is 28 feet (8.5 meters) and for the lower roof is 33 feet (10.1 meters). The distance W (horizontal distance between the eave and the ridge) will not be involved in the computation, since this is a flat roof. We have taken it as 33 feet (10.1 meters) also (even though it will not be involved in the computation). The intensity of ground snow load p_g = 25 pounds per square foot (1.2 kilonewtons per square meter). The risk category is II, therefore the importance factor I_S = 1.0.

Please note: with the current provisions of the code, the use of snow importance factor is no longer necessary because four separate maps are now provided for the four risk categories to select the ground snow load. We have indicated the importance factor here for those readers who may be using older versions of the code.

Furthermore, we take $C_s = C_e = C_t$ = 1.0. The building is located in a midwestern region, for the parameter W_2, use 0.45 for this paramter.

Determine the length of the drift w and the intensity of the snow load on the roof.

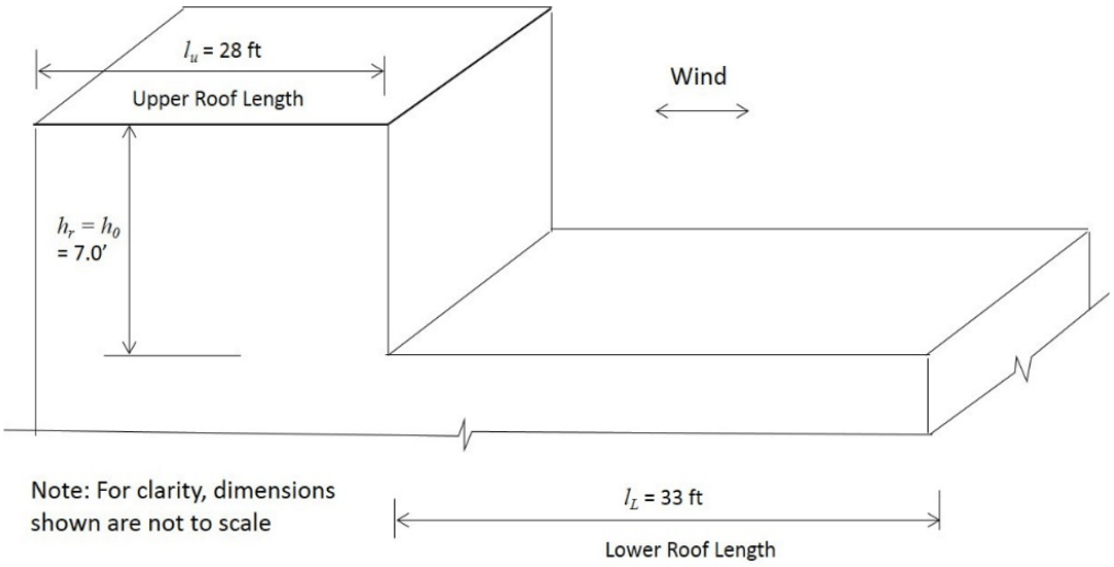


Figure E6.2a Roof configuration in Example 6.2

(This figure shows dimensions of a flat roof about 7 feet above a lower roof. The length of the upper roof is shown as 28 feet; the length of the lower roof is shown as 33 feet.)

Solution:

Using the ASCE-7 procedure, the snow density is

$\gamma = 0.13p_g + 14$ = 0.13x25 + 14 = 17.25 pounds per cubic foot (276.3 kilograms per cubic meter), which is less than the limit of 30 pounds per cubic foot (480.6 kilograms per cubic meter), Thus γ is taken as 17.25 pounds per cubic foot or 276.3 kilograms per cubic meter.

The unbalanced clear height is $h_c = h_r - h_b$, however, h_b is equal to p_s/γ, in which $p_s = C_e p_f$ and $p_f = 0.7C_sC_tp_g$, thus

$p_s = p_f = 0.7C_eC_tp_g$ = 0.7x1x1x25 = 17.5 pounds per square foot, which is less than the limiting value of 20 pounds per square foot, therefore, p_s is taken as 20 pounds per square foot (0.96 kilonewtons per square meter).

h_b = 20/17.25 = 1.16 feet

$h_c = h_r - h_b$ = 7.0 – 1.16 = 5.84 feet

h_c/h_b = 5.84/1.16 = 5.04, which is larger than the limiting value of 0.2, thus snow drift must be considered for this roof.

Using l_U and l_L equal to 28 and 33 feet, respectively,

$h_{dL} = 1.5\sqrt{\frac{p_g^{0.74}l_U^{0.7}W_2^{1.7}}{\gamma}} = 1.5\sqrt{\frac{25^{0.74}28^{0.7}0.45^{1.7}}{17.25}}$ = 1.93 feet (0.59 meters) for leeward. Note that per ASCE, this need not exceed 60% of the length of the lower roof.

And,

$h_{dW} = 0.75[1.5\sqrt{\frac{p_g^{0.74}l_U^{0.7}W_2^{1.7}}{\gamma}}]$ = 1.54 feet (0.47 meters) for windward. Note that in this equation the length of the lower roof is used

for L_U.

h_d is the larger of the two, thus h_d = 1.93 feet (0.59 meters).

Since h_d is smaller than h_c, thus

$w = 4h_d$ = 4x1.93 = 7.72 feet (4.65 meters), and snow drift load $p_d = \gamma h_d$ = 17.25x1.93 = 33.3 pounds per square foot, with a triangular distribution over the length w (in addition to the uniform snow load of 20 pounds per square foot). Figure E6.2b shows the snow drift profile.

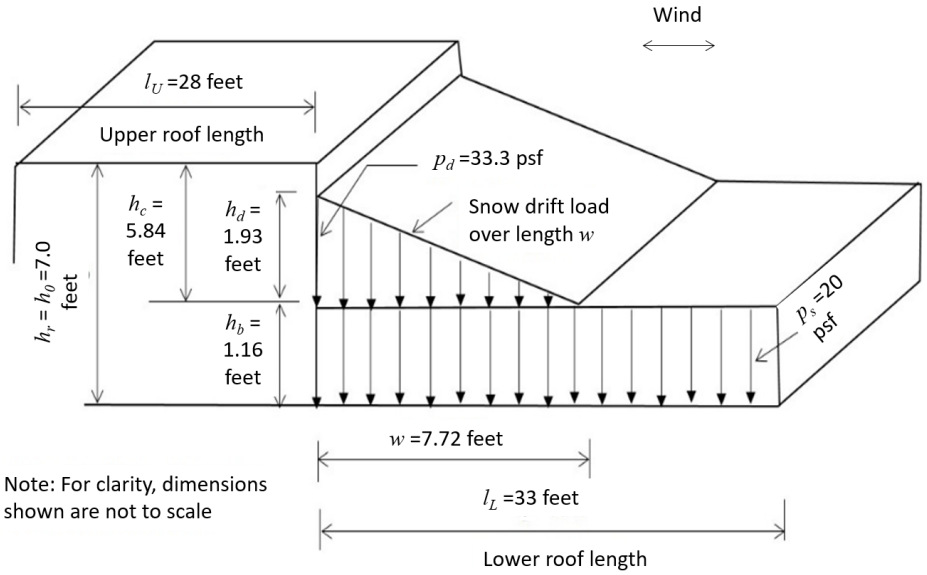


Figure E6.2b Snow drift profile for Example 6.2

(This figure shows a profile of the wind load on the roof with all computed values and dimensions shown.)

Example 6.3 (Refer to Section 6.3)

In this example, the roof lower length is the same as the one in Example 6.2. There is a parapet wall of 2 feet width and 3.5 feet height (i.e., $h_r = h_0$ equal to 3.5 feet and l_U equal to 2 feet), as shown in Figure E6.3a. The ground snow information, building category and other parameters are identical with those in Example 6.2. Compute the snow pressure resulting from the wind drift.

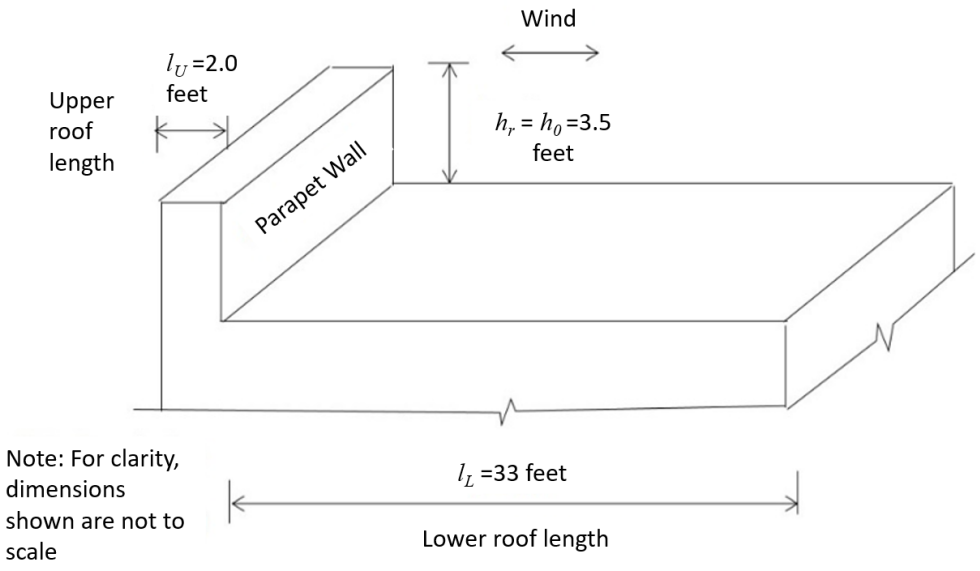


Figure 6.3a Roof configuration for Example 6.3

Solution:

As determined in Example 6.2, γ = 17.25 pounds per cubic foot and $p_s = p_f$ = 20 psf and $h - b$ is 1.16 feet.

However, h_c is different.

$h_c = h_r$ = 3.5 – 1.16 = 2.34 feet

h_c/h_b = 2.34/1.16 = 2.02, which is larger than 0.2, thus snow drift must be considered.

In this case l_U is equal to 2 feet, and, l_L, as in Example 6.2, is 33 feet. And also note that W_2 is equal to 0.45.

$h_{dL} = 1.5\sqrt{\frac{p_g^{0.74} l_U^{0.7} W_2^{1.7}}{\gamma}} = 1.5\sqrt{\frac{25^{0.74} 2.0^{0.7} 0.45^{1.7}}{17.25}}$ = 0.77 feet (0.23 meters) for leeward.

And

$h_{dW} = 0.75 \times 1.5\sqrt{\frac{p_g^{0.74} l_U^{0.7} W_2^{1.7}}{\gamma}} = 0.75 \times 1.5\sqrt{\frac{25^{0.74} 33^{0.7} 0.45^{1.7}}{17.25}}$ = 1.54 feet (0.0.47 meters) for windward.

h_d is larger than the two, thus, h_d = 1.54 feet, and since this is less than h_c,

$w = 4h_d$ = 4x1.54 =6.16 feet (1.88 meters)

and snow drift load $p_s = \gamma h_d$ = 17.25x1.54 = 26.6 pounds per square foot (2.0 kilonewtons per square meter)

With a triangular distribution over the length w (in addition to 20 pounds per square foot, which is uniform).

Example 6.4 (refer to Section 6.4)

The roof for a 40-unit apartment complex will need to be replaced. The owner is considering a green roof option. The roof area is 10,000 square feet. The following is the cost information for an intensive green roof (Option 1) and a conventional roof (Option 2)

Option 1: Intensive Green Roof

Initial cost: $35 per square foot

Annual repair and maintenance: $0.40 per square foot in the first year. This will be increased by $0.02 every year thereafter.

Major repair work at mid-life ($12,000). The midlife is 15 years.

Saving in energy consumption for the entire complex: $7,500 per year

Option 2: Conventional Roof

Initial cost: $9 per square foot

Annual repair and maintenance $0.10 per square foot in the first year. This will increase $0.02 per square foot every year thereafter.

No additional cost

Compare these two options for a service life of n = 30 year and an interest rate of i =0.05 per year.

Solution:

This is an engineering economic problem. We will use the total present worth P value for each option. The equations for computing present worth are provided in Chapter 9.

Future value F converted to present worth: $P = \frac{F}{(1+i)^n}$

End of period payment A converted to present worth: $P = A\frac{(1+i)^n-1}{i(1+i)^n}$

Gradient value G, which is the increase after year 1, converted to an equivalent present worth: $A_{Eq} = \frac{G}{i} - \frac{nG}{i}\left[\frac{i}{(1+i)^n-1}\right]$

And

$A_{Total} = A + A_{Eq}$

We will conduct all computations for one square of foot. Thus, return on energy saving per year is $0.75 and the major repair work at mid-life is $1.2 per square foot.

For option 1

P_1 = -35 (Initial cost)

$A_{Eq} = \frac{0.02}{0.05} - \frac{30\times0.02}{0.05}\left[\frac{0.05}{(1+0.05)^{30}-1}\right]$ = 0.219

A_{Total} = 0.40+0.219 =0.619

Then

$P_2 = A\frac{(1+i)^n-1}{i(1+i)^n} = 0.619\frac{(1+0.05)^{30}-1}{0.05(1+0.05)^{30}}$ = -9.52

$P_3 = \frac{F}{(1+i)^n} = \frac{1.20}{(1+0.05)^{15}}$ = -0.58 (converting repair cost at n = 15 to present worth)

$P_4 = A\frac{(1+i)^n-1}{i(1+i)^n} = 0.75\frac{(1+0.05)^{30}-1}{0.05(1+0.05)^{30}}$ = +11.53 (annual saving converted into present worth)

PW (for Option 1) = - (35.00+9.52+0.58) + 11.53 = - 33.57

For Option 2

P_1= -9 (Initial cost)

A_{Eq} = 0.219 and A_{Total} = 0.10 + 0.219 = 0.319

$P_2 = A\frac{(1+i)^n-1}{i(1+i)^n} = 0.319\frac{(1+0.05)^{30}-1}{0.05(1+0.05)^{30}}$ = -4.90

PW (for Option 2) = - (9.00+4.90) = - 13.90

Obviously, Option 2 is preferred.

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11.7: Illustrative Examples for Chapter 7

Example 7.1 (Refer to Section 7.2)

An angle 6-inch by 6-inch with one-half of an inch thickness is used as a shelf angle to support a 4-inch veneer wall of 13 feet and 4 inches (13.33 feet or 4.1 meters) in height. The cavity gap used is equal to 2 inches (51 millimeters). The spacing for the anchor S is taken as 4 feet (1.22 meters).

a) Determine whether the angle thickness is adequate.

b) Compute the additional tensile stress developed in the angle vertical leg.

Solution:

a) The effective length of the angle b is equal to 4 times 4.0 inches (16.0 inches or 0.41 meters). The angle selected has a total weight of 19.6 pounds per linear foot (286 newotons per meter). Thus q_a is equal to 19.6 divided by 2 or 9.8 pounds per linear foot (143 newtons per meter). The value of F_y is 36,000 pounds per square inch (248 megapascals); and the weight of a 4-inch (102-millimeter) veneer wall is 42 pounds per square foot (2.01 kilonewtons per square meter). With 2.0 inches (51 millimeters) of cavity, e is 3.625/2 + 2.0 + 0.5x1/2 or 4.06 inches (103 millimeters). Thus using Equation 7.2.2.15 given in Section 7.2 of Chapter 7, the minimum angle thickness t is

$t = \sqrt{\frac{8.4(w \times h + q_a)S \times e}{\phi F_y b}} = \sqrt{\frac{8.4 \times (42 \times 13.33 + 9.8) \times 4.0 \times 4.06}{0.9 \times 36{,}000 \times 16}}$ = 0.39 inches (10 millimeters)

Therefore, the selected angle is acceptable with one half an inch (13 millimeters) of thickness.

b) The factored additional tensile stress (using Equation 7.2.2.16 of Section 7.2) in the vertical leg of the angle is

$f_t = \frac{1.4(w \times h + q_a)S}{b \times t} = \frac{1.4 \times (42 \times 13.33 + 9.8) \times 4.0}{16.0 \times 0.5}$ = 399 pounds per square inch (2.75 megapascals)

Example 7.2 (Refer to Section 7.2)

In an application, the angle shown in Figure E2.5 (of Example 2.5) is used as a shelf angle to support a 4-inch veneer wall of 9.5 feet (2.9 meters) in height. The cavity gap used is 1 inch (25 millimeters). The spacing S for the anchors is taken as 1 foot (0.3 meters).

a) Determine whether the angle thickness is adequate.

b) Compute the deflection at the tip of the outstanding leg of the angle.

Solution:

a) Again, the effective length b of the angle is 16.0 inches. However, this value exceeds the anchor spacing of 1 foot (12 inches), thus we use b equal to 12 inches (0.3 meters). The angle selected has a total weight of 6.1 pounds per foot. Thus q_a is one half of 6.1 or 3.1 pounds per foot (45 newtons per meter). With 1 inch (25 millimeters) of cavity, e equals 3.625/2 + 1 + 0.5x0.25 or 2.94 inches (75 millimeters). Thus using Equation 7.2.2.15 (of Section 7.2 of Chapter 7), the minimum angle thickness is

$t = \sqrt{\frac{8.4 \times (42 \times 9.5 + 3.1) \times 1.0 \times 2.94}{0.9 \times 36{,}000 \times 12}}$ = 0.16 inches (4 millimeters)

Therefore, the selected angle is acceptable with a quarter of an inch (6 millimeters) of thickness.

b) The moment of inertia for b equal to 12 inches (0.3 meters) is

$I = \frac{12.0 \times 0.25^3}{12}$ = 0.0156 inches to the power of 4 (6,493 millimeters to the power of 4)

and the total weight W is (42x9.5+3.1)x1.0, which is 402.1 pounds (1.8 kilonewtons). Using the equation given for the deflection of the tip of the angle in Section 7.2.2 of Chapter 7,

$y = \frac{W \times e^2}{6EI}(3a - e) = \frac{402.1 \times 2.94^2 (3 \times 4.0 - 2.94)}{6 \times 29{,}000{,}000 \times 0.0156}$ = 0.01 inches (0.3 millimeters)

The specifications indicate a limit of $L/600$ or 3-8th of an inch (9.5 millimeters) for the combination of live and dead loads for structural systems supporting the veneer wall and the shelf angle, in which L is the spacing between columns at the perimeter of the building, WSCPA (2011). However, no specific limit for the deflection of the tip of a shelf angle is specified.

Example 7.3 (Refer to Section 7.3)

Figure E7.3a shows the details of a shelf angle (6-inch by 6-inch with 3 quarters of an inch thickness) attachment to a concrete spandrel beam. The following information is provided:

The angle supports a wall of 14 feet (4.3 meters) with a nominal wall thickness T_E equal to 4.0 inches (102 millimeters). Anchors are spaced at 2 feet (610 millimeters) from one another. The edge distance of anchors from the top, bottom and side of the beam is 8 inches (203 millimeters). The information on the anchor is provided below:

Anchor type: ASTM F1554 Grade 36

f_{uta} = 58,000 pounds per square inch (400 megapascals), specified tensile strength of anchor steel

f_{ya} = 105,000 pounds per square inch (724 megapascals), specified yield strength of anchor steel

$A_{se,N}$ = 0.142 square inches (92 square millimeters), anchor effective cross-sectional area

A_{brg} = 0.291 square inches (188 square millimeters), anchor bearing cross-sectional area

d_a = 0.5 inches (12.7 millimeters), anchor nominal diameter

h_{ef} = 4 inches, anchor embedment length

The compressive strength of the concrete f'_c = 4,000 pounds per square inch (27.6 megapascals), and the wall load eccentricity e is 4.06 inches (103 millimeters). The structure is in a non-seismic area.

Using the ACI (2014) provision for anchor design, check whether the designated anchor is acceptable.

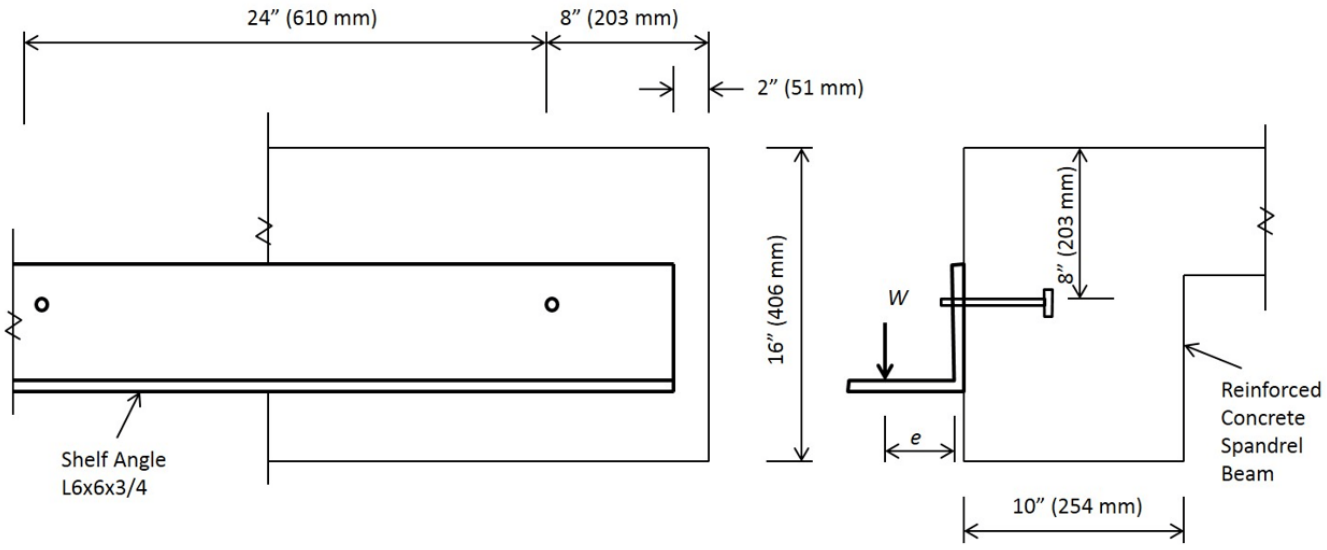


Fig. E7.3a Anchor configuration in Example 7.3

(On the left, this figure shows the longitudinal detail of an angle attachment to a concrete beam. On the right, a cross-section of the angle and its attachment to the concrete beam is shown with dimensions marked on each detail.)

Solutions:

1. Load computation

(a) Wall weight W

The shelf angle effective length b is taken as $4T_E$ but not to exceed the spacing S. Thus b is 16 inches (406 millimeters).

Self-weight of the angle = 19.6 pounds per foot

Recall that for the weight computation, we use the anchor spacing S, which is 2 feet, thus

W = 14.0x(42)x(2.0 feet) + (19.6/2)x(2.0 feet) = 1,195.6 pounds (5.3 kilonewtons) for computation of tension in the anchor

W = 14.0x(42)x(2.0 feet) + (19.6)x(2.0 feet) = 1,215.2 pounds (5.4 kilonewtons) for computation of shear in the anchor

Notice that in these calculations we are using 42 pounds per square foot for the wall weight, one half of the self-weight of the angle for the anchor tension, and the entire self-weight of the angle for anchor shear force computation.

(b) Computation of tension and shear forces in the anchor

The lever arm between tension in the anchor and compression force on concrete beam y is equal to 3.0 inches minus one half of the angle thickness (0.75 inches), or 2.625 inches (67 millimeters). Thus,

Anchor tension = $\frac{W \times e}{y}$ = (1,195.6x4.06/2.625) = 1,849.2 pounds (8.23 kilonewtons)

Anchor shear =W = 1,215.2 lbs (5.41 kilonewtons)

Since W is made entirely of the dead load, the load factor is 1.4 and thus

N_{ua} = 1.4x1,849.2 = 2,589.0 pounds (11.52 kilonewtons)

V_{ua} = 1.4x1,215.2 = 1,701.3 pounds (7.57 kilonewtons)

2. Strength computation for tension

(a) Steel tension

Equation 17.4.1.2 of ACI: $N_{sa} = A_{se,N} \times f_{uta}$

In which, N_{sa} is the steel strength in tension and f_{uta} is the minimum of $1.9f_{ya}$ or 125,000 pounds per square inch. In this case, f_{uta} will then be 58,000 pounds per square inch (400 megapascals), and

N_{sa} = 0.142x58,000 = 8,236 pounds (36.64 kilonewtons)

ϕN_{sa} = 0.75x8,236 = 6,177 pounds (27.48 kilonewtons)

Strength utilization ratio = 2,589.0/6,177 = 0.419

(b) Concrete breakout strength in tension

We need to determine whether anchor groups should be considered. For concrete breakout strength in tension, critical spacing is $3h_{ef}$ or 3 times 4.0 resulting is 12.0 inches (305 millimeters). Since S is 24 inches, hence larger than $3h_{ef}$, the anchor group effect is not critical. Thus, the anchor strength calculation will be done for a single anchor only.

Equation 17.4.2.1a of ACI: $N_{cb} = \frac{A_{Nc}}{A_{Nc0}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$

In which

A_{Nc} = Projected concrete failure area for a single anchor or group of anchors. For a single anchor, this is approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerline of the anchor (see Figure E7.3b). Depending upon whether the edge distances c_{a1} and c_{a2} are smaller than $1.5h_{ef}$, five different situations are shown in Figure E7.3b for the computation of the area A_{Nc}. The edge distances c_{a1} and c_{a2} are measured in the x and y-directions, respectively. Notice that the area A_{Nco} corresponds to the case (b) shown in Figure E7.3b and is the projected failure area of a single anchor with edge distances $3h_{ef}$. Thus, this area is computed as

Equation 17.4.2.1c of ACI: $A_{Nc0} = 9h_{ef}^2$

In general, A_{Nc} is smaller or equal to A_{Nco} for a single anchor and A_{Nc} is smaller or equal to nA_{Nco} for anchor group, in which n is the number of anchors in the group.

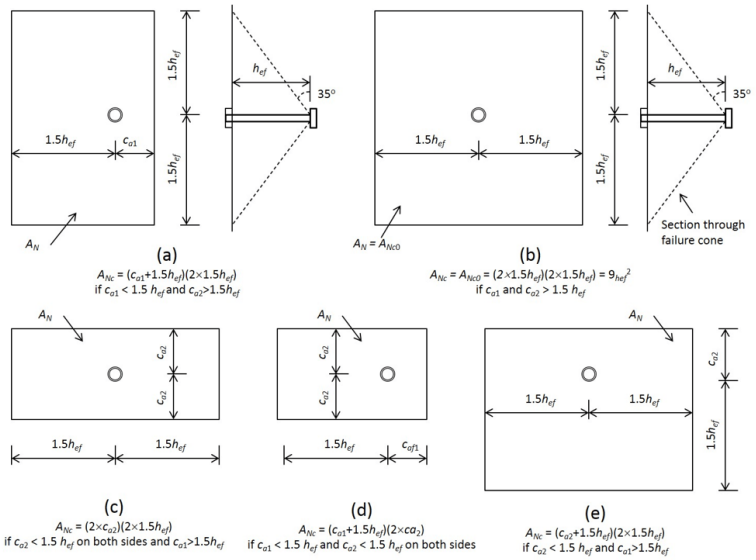


Figure E7.3b Various anchor configurations for computing A_{Nc} in Example 7.3

(This figure shows six different configurations for computing the projected failure area involved in computing the breakout strength in tension for an anchor embedded in concrete.)

In this example, c_{a1} and c_{a2} each is 8 inches, which is larger than $1.5h_{ef}$ (6 inches), thus $A_{Nc} = A_{Nc0} = 9 \times 4.0^2$, which is 144 square inches (92,903 square millimeters). The basic concrete breakout strength of a single anchor in tension in cracked concrete is then computed as follows.

Equation 17.4.2a of ACI: $N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$

in which, λ_a is equal to 1 for normal weight concrete, and K_c is equal to 24 for cast-in anchors. Furthermore

$N_b \leq 16\lambda_a \sqrt{f'_c} h_{ef}^{5/3}$ in situations when h_{ef} is between the limits of 11 inches (279 millimeters) and 25 inches (635 millimeters), inclusive of these two limits. Since in this problem h_{ef} is equal to 4 inches, thus

$N_b = 24 \times 1.0 \times \sqrt{4,000} \times (4.0)^{1.5}$ = 12,143 pounds (54.0 kilonewtons)

Modification factors:

$\psi_{ec,N}$ is the modification factor for anchor group loaded eccentrically in tension, where according to Equation 17.4.2.4 of ACI:

$\psi_{ec,N} = \frac{1}{(1+\frac{2e'_N}{3h_{ef}})}$

where, e'_N is the group anchor load eccentricity. Since group anchor effect is not critical in this example,

$\psi_{ec,N}$ = 1.0

$\psi_{ed,N}$ = is the modification factor for edge effect, and

$\psi_{ed,N}$ = 1.0, if $c_{a,min} \geq 1.5h_{ef}$, and per Equations 17.4.2.5a & b of ACI,

$\psi_{ed,N} = 0.70 + 0.3\frac{c_{a,min}}{1.5h_{ef}}$ if $c_{a,min}$ is less than $1.5h_{ef}$.

In this example, since $c_{a,min}$ is equal to 8 inches and alrger than $1.5h_{ef}$ (6 inches), $\psi_{ed,N}$ is equal to 1.0.

$\psi_{c,N}$ is the modification factor for non-cracking in service load. In this example, cracked concrete is assumed, thus this factor is taken as 1.0

$\psi_{cp,N}$ is the modification factor for post-installed anchors. In this example, anchors are cast-in, thus this factor is also taken as 1.0.

It is noted that if post-installed anchors are involved, per ACI

$\psi_{cp,N}$ is equal to 1.0, if $c_{a,min}$ is greater than or equal to c_{ac}, and

$\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \geq 1.5\frac{h_{ef}}{c_{ac}}$, if $c_{a,min}$ is less than c_{ac}

In which

c_{ac} is critical edge distance, and per Section 17.7.6 of ACI, is not to be taken less than,

$2h_{ef}$ for adhesive anchors,

h_{ef} for undercut anchors,

$4h_{ef}$ for torque-controlled expansion anchors, and,

$4h_{ef}$ for displacement-controlled anchors.

Considering all the modification factors, thus

N_{cb} = (144.0/144.0)x1.0x1.0x1.0x12,143 = 12,143 pounds (54.0 kilonewtons)

Since no supplemental reinforcement is used, Condition B as specified in Table 7.3 (of Chapter 7) governs and the resistance reduction factorϕ is 0.7. Thus

ϕN_{cb} = 0.7x12,143 = 8,500.1 pounds (37.8 kilonewtons)

Strength utilization ratio = 2,589/8500.1 = 0.305

(c) Pullout strength in tension

Per Equation 17.4.3.1 of ACI,

$N_{pn} = \psi_{c,p}N_p$

In which,

$\psi_{c,p}$ is equal to 1.0 for cracked concrete (which is the situation in this example) and 1.4 if analysis shows concrete is non-cracked.

Furthermore, per Equation 17.4.3.4 of ACI, for single-headed stud or headed bolt anchors

$N_p \leq 8A_{brg}f'_c$

Thus, N_p = 8x0.291x4,000 = 9,312 pounds (41.4 kilonewtons)

$N_{p,n}$ =1.0x9,312 = 9,312 pounds (41.4 kilonewtons)

with ϕ equal to 0.7,

$\phi N_{p,n}$ = 0.7x9,312 = 6,518.4 pounds (29.0 kilonewtons)

Strength utilization ratio = 2,589.0/6,518.4 = 0.397

(d) Side-face blowout strength in tension

In this example, since h_{ef} is 4 inches and is smaller than 2.5$c_{a,1}$ (20 inches), the side-face blowout mode of failure is not prevalent.

(e) Bond strength in tension

This mode of failure pertains to adhesive anchors; and as such, it is not prevalent in this example.

3. Strength computation for shear

(a) Steel element shear strength

V_{sa} is less than or equal to $A_{se,V}f_{uta}$ for cast-in headed stud anchor, and less than or equal to $0.6A_{se,V}f_{uta}$ for cast-in headed bolt and hooked bolt anchors and post-installed anchors where sleeves do not extend through shear planes. Note that (1) if sleeves extend through shear planes, V_{sa} will need to be determined by tests; and (2), if anchors are used with built-up grout pads, the nominal strength is multiplied by a factor of 0.8 (per Equations 17.5.1.2a & b of ACI). Furthermore, note that

$A_{se,V}$ is the effective cross-sectional area of the anchor in shear, which is 0.142 square inches (92 square millimeters) and f_{uta} is 58,000 pounds per square inch (400 megapascals), as determined earlier, thus

V_{sa} = 0.6x0.142x58,000 = 4,941.6 pounds (22.0 kilonewtons)

with ϕ equal to 0.65 for shear, ϕV_{sa} = 0.65x4,941.6 = 3,212.0 pounds (14.3 kilonewtons)

Strength utilization = 1,701.3/3,212.0 = 0.530

(b) Concrete breakout in shear

The concrete breakout strength in shear for a single anchor for shear perpendicular to the edge per Equation 17.5.2.1a of ACI is

$V_{cb} = \frac{A_{Vc}}{A_{Vc0}}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b$

If shear is parallel to the edge, V_{cb} from the above equation is multiplied by 2. In our case, shear is perpendicular to the edge and $\psi_{ed,V}$ is equal to 1, therefore, we can use the above equation, in which

V_b is the basic concrete breakout strength in shear for a single anchor. For cracked concrete, it is the smaller of the values obtained from the two equations given below.

Equation 17.5.2.2.2a of ACI: $V_b = \left[7(\frac{\ell_e}{d_a})^{0.2}\sqrt{d_a}\right]\lambda_a\sqrt{f'_c}(c_{a1})^{1.5}$

in which,

ℓ_e is the load bearing length of the anchor for shear, and it is equal to h_{ef} for anchors with a constant stiffness over the full length such as headed stud anchors or post-installed anchor with one tubular shell over the full length of the embedded length; and equal to 2d_e for torque-controlled expansion sleeve with a distance sleeve separated from expansion sleeve. In all cases, ℓ_e cannot exceed 8d_a.

Equation 17.5.2.2.2b of ACI: $V_b = 9\lambda_a\sqrt{f'_c}(c_{a1})^{1.5}$

For cast-in headed studs, headed bolts or hooked bolts that are continuously welded to steel attachments having a minimum thickness larger than or equal to 3-8th of an inch (9.5 millimeters) and d_a/2, V_b is the smaller of the value from the following two equations.

Equations 17.5.2.3 of ACI: $V_b = \left[8(\frac{\ell_e}{d_a})^{0.2}\sqrt{d_a}\right]\lambda_a\sqrt{f'_c}(c_{a1})^{1.5}$

Equation 17.5.2.2b of ACI: $V_b = 9\lambda_a\sqrt{f'_c}(c_{a1})^{1.5}$

Provided that

(i) For a group of anchors, the strength is determined for a row farthest from the edge;

(ii) Anchor spacing is not less than 2.5 inches (64 mm); and,

(iii) Reinforcement is provided at the corners if c_{a2} is less than or equal to 1.5h_{ef}.

Furthermore,

A_{Vc} is the projected area of failure surface on the side of the concrete member at its edge. A_{Vc} can be taken as the area of the base of a truncated half pyramid projected on the side face of the member where the top of the half-pyramid is given by the axis of the anchor row selected as critical. The value of c_{a1} is taken as the distance from edge to this axis. Note that A_{Vc} is less than or equal to nA_{Vc0}, where n is number of anchors in the group.

A_{Vc0} is the projected area of a single anchor in a deep member with a distance from edges larger than or equal to 1.5c_{a1} in the direction perpendicular to the shear face. This area can be evaluated based on a half-pyramid with a side length parallel to the edge of 3c_{a1} and a depth of 1.5c_{a1}. Note that deep member is considered when the thickness, h_a is larger than 1.5c_{a1}. Accordingly, and per Equation 17.5.2.1c of ACI

$A_{Vc0} = 4.5(c_{a1})^2$

In this problem, h_a is equal to 10 inches (254 millimeters) and thus less than 1.5c_{a1} (which is 12.0 inches or 305 millimeters). Therefore, for computing A_{Vc}, since c_{a2} is equal to 8.0 inches and is less than 1.5c_{a1} inches, the base of the pyramid extends c_{a2} on each side of the anchor axis. This means the base of the pyramid has a dimension equal to 2c_{a2} as shown in Figure E7.3c. Also, it noted that the minimum of h_a (10 inches or 254 millimeters) and 1.5c_{a1} (12.0 inches or 305 millimeters) is used as the pyramid base dimension along the side of the concrete spandrel beam. Thus,

$A_{Vc} = [min(h_a, 1.5c_{a1})] \times (2c_{a2})$ = 10.0x(16.0) = 160 square inches (103,226 square millimeters)

A_{Vc0} = 4.5 $(8)^2$ = 288 square inches. Furthermore, the value of ℓ_e is taken as the smaller of h_{ef} and 8 times the diameter of the anchor (which is 0.5 inches). This results in ℓ_e equal to 4 inches (102 millimeters). As a result,

$V_b = \left[7(\frac{4.0}{0.5})^{0.2}\sqrt{0.5}\right] \times 1.0\sqrt{4,000}(8.0)^{1.5}$ = 10,736.6 pounds (47.8 kilonewtons)

And

$V_b = 9 \times 1.0\sqrt{4,000}(8.0)^{1.5}$ =12,879.8 pounds (57.3 kilonewtons)

Thus the value of V_b is taken as 10,736.6 pounds (47.8 kilonewtons).

and V_{cb} can be computed as described below.

Since, c_{a2} (8.0 inches) is less than 1.5c_{a1} (12.0 inches),

$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}}$ =0.7 + 0.3x8.0/12.0 = 0.9

$\psi_{c,V}$ =1.0, since concrete is assumed cracked,

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} > 1$, if $h_a < 1.5c_{a1}$

Since, in this problem, h_a (10 inches) is less than 1.5c_{a1} (12 inches), thus

$\psi_{h,V} = \sqrt{\frac{12.0}{10}}$ = 1.1

Finally,

V_{cb} = (160/288)x0.9x1.0x1.1x10,736.6 = 5,905.1 pounds (26.3 kilonewtons), and

ϕV_{cb} = 0.7x5,905.1 = 4,133.6 pounds (18.4 kilonewtons)

Strength utilization = 1,701.3/4,133.6 = 0.412.

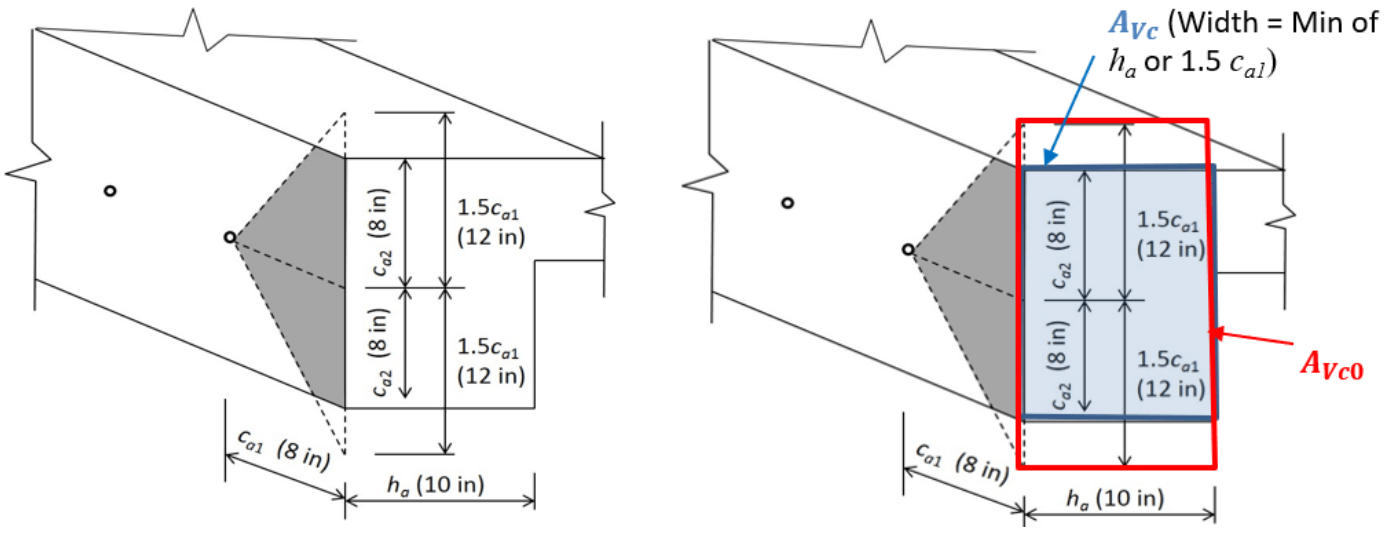


Figure E7.3c Computation of A_{Vc} for Example 7.3

(This figure illustrates how the portion of that is involved in the equation for the shear resistance through concrete breakout in shear can be computed. The dimensions of a pyramid that separates from the concrete during mode of failure resulting from the concrete breakout in shear are marked on the illustrations.)

(c) Concrete pryout strength

For a single anchor, per Equation 17.5.3.1a of ACI

$V_{cp} = k_{cp}N_{cp}$

Where for cast-in expansion and undercut anchors,

$N_{cp} = N_{cb}$ =12,413 pounds and k_{cp} =1.0, if h_{ef} is less then 2.5 inches, otherwise, k_{cp} is equal to 2.0, which is the value to be used in this problem, since h_{ef} is equal to 4.0 inches.

Thus, V_{cp} = 2.0x12,143 = 24,286 pounds (108 kilonewtons), and ϕV_{cp} = 17,000.2 pounds (75.6 kilonewtons).

Strength utilization = 1,701.3/17,000.2 = 0.100.

4. Interaction of tension and shear

The strength utilization factors for all modes considered are now used in verifying the anchor design using one of the two interaction equations explained in Section 7.2 of Chapter 7. It is noted that the interaction is done for tension and shear (that act in perpendicular direction with respect to one another.

The ratio of factored load to factored strength (strength utilization) for all modes of failure is summarized below.

Mode of Failure and Strength Utilization

Mode of Failure	Strength Utilization
Steel Tension	0.419 (maximum for tension)
Breakout concrete in tension	0.305
Pullout	0.397
Concrete side-face blowout	Not applicable
Bond in tension	Not applicable
Steel shear	0.530 (maximum for shear)
Concrete breakout in shear	0.412
Concrete pryout	0.1

$(\frac{N_{ua}}{\phi N_n}) + (\frac{V_{ua}}{\phi V_n})$ = 0.419 + 0.530 = 0.949, which is smaller than the limiting value of 1.2, alternatively,

$(\frac{N_{ua}}{\phi N_n})^{5/3} + (\frac{V_{ua}}{\phi V_n})^{5/3}$ = $(0.419)^{(5/3)}$ +$(0.530)^{(5/3)}$ = 0.582, which is less than the limiting value of 1.0.

Therefor the anchor design is acceptable.

Example 7.4 (Refer to Section 7.3.2)

The shelf angle in Example 7.3 (see Figure E7.3a) is being considered for attachment to a masonry reinforced grouted brick load-bearing wall. The masonry strength f'_m is equal to 2,000 pounds per square inch (6.9 megapascals). The anchor spacing is 2.0 feet (0.6 meters), the anchors used are headed ASTM 307 with one half an inch in diameter (13 millimeters) with a nominal cross-sectional area of 0.196 and an effective area A_b of 0.142 square inches (92 square millimeters). The embedment length l_b is 6 inches (152 millimeters) and the edge distance l_{be} is 8 inches (203 millimeters). The wall thickness is 12 inches (305 millimeters).

a) Compute the allowable axial tension and shear loads of the anchor and determine whether the anchor size selected is satisfactory using the ASD method.

b) Compute the design axial load and shear load strength of the anchor and determine whether the anchor size selected is satisfactory using the LRFD method.

Solution:

(a) For an A307 anchor, f_y is equal to 36,000 pounds per square inch (248.2 megapascals). Following the equations given in Section 7.3.2 of Chapter 7,

$A_{pt} = \pi l_b^2 = \pi \times (6.0)^2$ = 113.1 square inches (72,968 square millimeters)

$A_{pv} = \frac{\pi l_{bc}^2}{2} = \frac{\pi \times (8.0)^2}{2}$ = 100.5 square inches (64,839 square millimeters)

The allowable tensile and shear load values can be computed as follows:

$B_{ab} = 1.25 A_{pt} \sqrt{f'_m} = 1.25 \times 113.1 \times \sqrt{2,000}$ = 6,322.5 pounds (28.1 kilonewtons)

$B_{as} = 0.6 A_b f_y = 0.6 \times 0.142 \times 36,000$ = 3,067.2 pounds (13.6 kilonewtons)

B_a is selected as the smaller of the two values, thus

B_a = 3,067.2 pounds (13.6 kilonewtons)

For shear, the allowable strength is the smallest of the following four values.

$B_{vb} = 1.25 A_{pv} \sqrt{f'_m} = 1.25 \times 100.5 \times \sqrt{2,000}$ = 5,618.1 pounds (25.0 kilonewtons)

$B_{vc} = 350 \sqrt[4]{f'_m A_b} = 350 \sqrt[4]{2,000 \times 0.142}$ =1,436.8 pounds (6.4 kilonewtons)

$B_{vpry} = 2.0 B_{ab}$ = 12,641.0 pounds (56.2 kilonewtons)

$B_{vs} = 0.36 A_b f_y = 0.36 \times 0.142 \times 36,000$ =1,840.3 pounds (8.3 kilonewtons)

Thus B_v is equal to 1,436.8 pounds (6.4 kilonewtons).

From Example 7.3, the axial load and shear forces on the anchors are computed as 1,849.2 and 1,215.2 pounds. Thus, for the combined effect of axial load and shear

(1,849.2/3,067.2)+(1,215.2/1,436.8) = 1.45

This value is greater than 1, thus design is not acceptable per ASD, and a larger diameter anchor is needed.

b) With the LRFD method, a load factor of 1.4 is applied to the applied loads. The nominal values for axial load and shear strength are computed as follows.

$B_{anb} = 4.0 A_{pt} \sqrt{f'_m} = 4.0 \times 113.1 \times \sqrt{2,000}$ = 20,232 pounds (90.0 kilonewtons)

$B_{ans} = A_b f_y = 0.142 \times 36,000$ = 5,112 pounds (23.0 kilonewtons)

Thus, the nominal axial load strength B_{an} is 5,112 pounds (23.0 kilonewtons)

The nominal shear strength B_{vn} is the smallest of the following four values.

$B_{vnb} = 4.0 A_{pv} \sqrt{f'_m} = 4 \times 100.5 \times \sqrt{2,000}$ = 17,978 pounds (80.0 kilonewtons)

$B_{vnc} = 1,050 \sqrt[4]{f'_m A_b} = 1,050 \sqrt[4]{2,000 \times 0.142}$ = 4,310 pounds per square inch (19.2 kilonewtons)

$B_{vnpry} = 2.0 B_{anb}$ = 40,464 pounds (180.0 kilonewtons)

$B_{vns} = 0.6 A_b f_y = 0.6 \times 0.142 \times 36,000$ = 3,067 pounds (13.8 kilonewtons)

Thus, the nominal shear strength B_{vn} (smallest of the four values) is 3,067 pounds (13.8 kilonewtons).

The capacity reduction factors are 0.9 and 0.8 for axial tension and shear, respectively, thus the combination of the axial tension and shear results in

(1.4x1,849.2/(0.9x5,112)+(1.4x1,215.2/0.8x3,067) = 1.26 (larger than 1.0)

Design is not acceptable per LRFD, and a larger diameter anchor will be needed.

Example 7.5 (Refer to Section 7.3.3)

An angle 6-inch by 6-inch by 3 quarters of an inch thick and a span length of 11.25 feet (3.4 meters) is used as a shelf angle. The connecting bolts at each end are 2 A325 bolts of 5-8th of an inch (16 millimeters) in diameter. The support is a flat plate welded vertically to the steel column. The eccentricity e (see Figure E7.5) for the weld design is 4.8 inches (122 millimeters).

a) Determine whether the angle size is acceptable for 10.5 feet (3.2 meters) of brick masonry of 42 pounds per square foot (205.1 kilograms per square meter) density.

b) Determine whether the bolts are adequate.

c) Design the weld using the AISC design provisions.

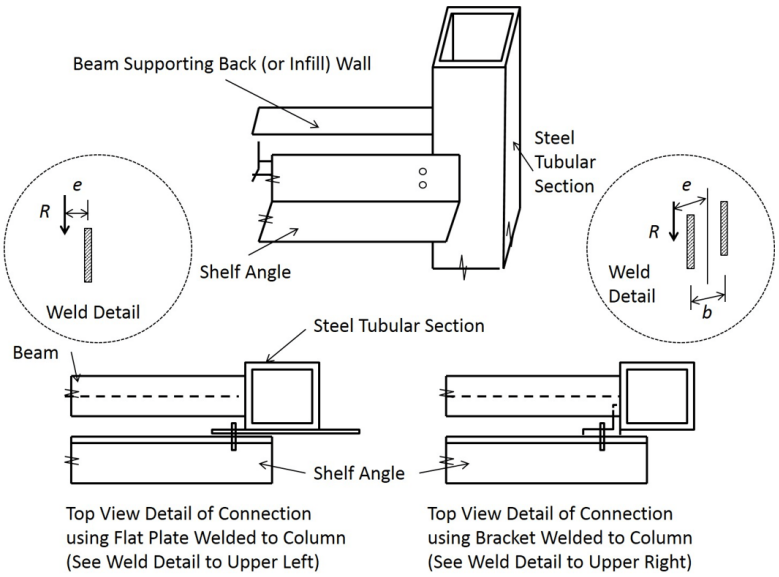


Figure E7.5 Shelf angle supported by steel columns for Example 7.5

(This figure shows an isometric detail of a shelf angle attached to a tubular steel column. Below this image, on the left, a cross-section of the detail, viewed from the top, is shown with a flat plate welded to the column and bolted to the shelf angle. On the right, the same cross-section is shown, however, the shelf angle connection to the column is via an angle-bracket that is welded to the column and bolted to the shelf angle.)

Solution:

a) The angle properties are obtained from the AISC manual and summarized below.

Section modulus: S_x equals 6.64 cubic inches (108,810 cubic millimeters)

Cross-sectional area: A equals 8.64 square inches (5,574 square millimeters)

Moment of inertia: I_x equals 28.1 inches to the power of 4 (11.7 times 10 to the power of 6 with a unit of millimeter to the power of 4)

Thickness: t is 0.75 inches (19 millimeters)

Width of the angle: b is 6 inches (152 millimeters)

We first need to determine the bending capacity of the angle. The yielding bending strength is

$M_y = S_X F_y$, and the nominal flexural strength is

$M_n = 1.5M_y$ = 1.5x(6.64x36.0) = 358.6 inch.kips (40.5 meter.kilonewtons)

Lateral-torsional buckling limit state, per AISC, for a single angle under bending about a geometric axis with no lateral torsional restraints is obtained from the following equation.

$M_y = 0.80F_yS_x$ = 0.80x36.0x6.64 = 191.2 inch.kips (21.6 meter.kilonewtons)

We also need to compute the critical bending moment resistance M_{cr}. For the bending moment about one of the geometric axes of an equal-leg angle with no axial compression, and no lateral-torsional restraint and maximum compression at the toe,

C_b is equal to 1.14 (lateral restraints at ends only), and

$M_{cr} = \frac{0.58Eb^4tC_b}{L_b^2}\left[\sqrt{1+0.88\left(\frac{L_bt}{b^2}\right)^2}-1\right]$

Thus

$M_{cr} = \frac{0.58\times(29{,}000)\times(6.0)^4\times0.75\times1.14}{(11.25\times12)^2}\left[\sqrt{1+0.88\left(\frac{11.25\times12\times0.75}{6.0^2}\right)^2}-1\right]$ = 1,862.8 inch.kips (210.5 meter.kionewtons)

The ratio M_y/M_{cr} is 191.2 divided by 1,862.8 and is equal to 0.103. This is less than 1.0, therefore, per AISC, the nominal capacity is computed as follows

$M_n = \left[1.92-1.17\sqrt{M_y/M_{cr}}\right]\times M_y$ but not to exceed $1.5M_y$

(Note: in this equation, M_y is 191.2 inch.kips)

$M_n = \left[1.92-1.17\sqrt{0.103}\right]\times 191.2$ = 295.3 inch.kips (33.4 meter.kilonewtons)

This value is larger than $1.5M_y$ = 1.5x191.2 = 286.8 inch.kips. Thus M_n = 286.8 inch.kips (32.4 meter.kilonewtons)

Finally, we must compute the moment capacity based on the leg local buckling effect. Per AISC, since the toe of the leg is in compression, the slenderness of the leg in compression is

$\lambda = b/t$ = 6.0/(0.75) = 8.0

The limiting non-compact slenderness ratios from AISC are

$\lambda_p = 0.54\sqrt{E/F_y} = 0.54\sqrt{29,000/36.0}$ =15.3

$\lambda_r = 0.91\sqrt{E/F_y} = 0.91\sqrt{29,000/36.0}$ =25.8

Since λ is less than λ_p, thus the leg is compact in flexure, and as such, the limit state for local buckling does not apply.

Thus, the torsional buckling limit state (which has a smallest bending capacity) controls, and

M_n = 286.8 inch.kips (32.4 meter.kilonewtons)

Using the LRFD approach, ϕ is 0.9; and the factored capacity is

ϕM_n = 0.9x286.8 = 258.1 in.kips = 21.51 foot.kips (29.2 meter.kilonewtons)

The factored applied load for a veneer height of h equal to 10.5 feet is

q_u = 1.4x42.0x10.5 = 617.4 pounds per foot (9.0 kilonewtons per meter)

$M_u = q_uL^2/8 = 617.4\times(11.25)^2/8$ = 9.77 foot.kips (13.2 meter.kilonewtons).

This is smaller than the bending resistance of 21.51 foot. kips (29.2 meter.kilonewtons), thus design is acceptable for bending.

However, we must make sure the deflection is also acceptable. The deflection is computed for the service load (in other words,

we do not use any load factor for computing the deflection).

q = 42.0x10.5 = 441 pounds per foot (6.4 kilonewtons per meter)

$y = \frac{5qL^4}{384EI} = \frac{5\times(441/12)\times(11.25\times12)^4}{384\times29{,}000{,}000\times28.1}$ = 0.20 inches (5 mm).

This value is less than the limiting vales of $L/600 = (11.25 \times 12)/600$ = 0.225 inches (5.7 millimeters) and 3-8th of an inch (9.5 millimeters), thus deflection is also acceptable.

b) The factored end reaction R is

R = 617.4x11.25/2 = 3,473 pounds (15.4 kilonewtons)

The design shear strength for 5-8th of an inch (15.9-millimeter) diameter bolt is

$\phi_v r_n$ = 11,000 pounds (48.9 kilonewtons) for each bolt per AISC, thus bolts are adequate.

c) The support plate has one vertical strip of weld with an eccentricity of the load being 4.8 inches. We use a length l as 6 inches for the weld. Thus, using the AISC weld tables (Table 8-4 of AISC), a 2-16th of an inch weld size and E70 electrode, the weld capacity (if there were two strips of weld) is

$\phi R_n = 0.75 \times CC_1Dl$ (for two strips of weld)

in which, C is 1.56 from weld design tables in AISC for the ratio e/l of 4.8/6.0 = 0.8, C_1 is 1 (for E70 electrode; and D is equal to 2 (since weld size is 2-16th of an inch). Thus

$\phi R_n = 0.75 \times 1.56 \times 1 \times 2 \times 6.0$ = 14.04 kips = 14,040 pounds (62.5 kilonewtons)

However, since we have one strip of weld, the factored resistance is one half of this value or 7,020 (31.2 kilonewtons). This is larger than the applied load of 3,473 pounds (15.4 kilonewtons), thus weld design is acceptable.

Example 7.6 (Refer to Section 7.4)

An angle is used as a lintel to support a brick masonry façade of 42 pounds per square foot (205.1 kilograms per meter) density. The brick façade above the lintel is built in place and has a height of 7.5 feet (2.3 meters). The clear span of the lintel is 8 feet (2.4 meters). Design an appropriate A36 steel angle for this lintel. There are no superimposed loads on the lintel.

Solution:

Based on the information provided, considering the fact that the masonry is built in place, the in-plane arching action is formed; and as such, a triangular load can be used. For a simply-supported beam with a triangular load of q at its peak (see Figure E7.6), the following equations are used for the bending moment, shear and deflection.

Bending moment: $M = qL^2/12$

Shear: $V = qL/4$

Deflection: $y = \frac{qL^4}{120EI}$

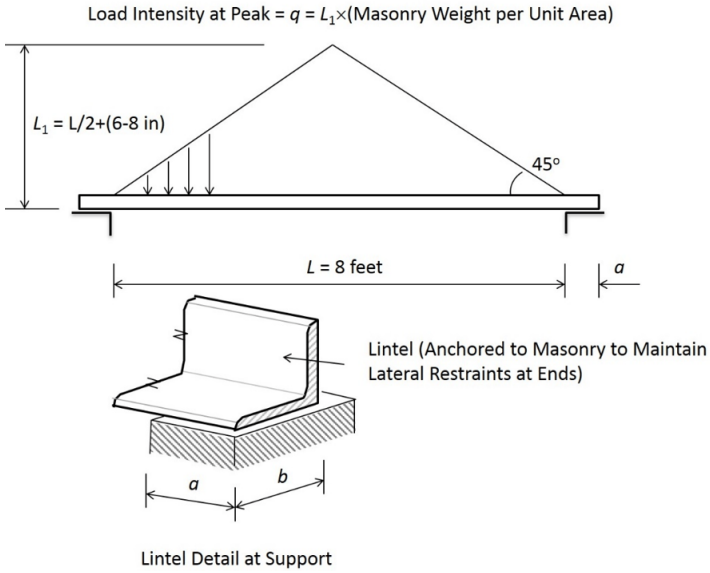


Figure E7.6 Details of the lintel in Example 7.6

(This figure shows a simply-supported beam with a triangular load on it. A detail is shown below the beam and depicts the support areas marked with dimensions *a* and *b*.)

As seen in Figure E7.6, the lintel at its ends has a support area equal to *a* times *b*. The reaction R is distributed over this area resulting in a compressive stress f_a on the masonry,

where

$f_a = \frac{R}{a \times b}$

For the strength computation, the provisions of AISC (2017) are used with lintel considered restrained laterally at the ends only.

In this example, we use $L_1 = L/2 + 8$ = 8x12/2 +8 = 56 inches = 4.67 feet (1.4 meters).

Service load: q = 4.67x42.0 = 196.1 pounds per foot (2.9 kilonewtons per meter)

Factored load: q_u = 1.4x196.1 = 274.6 pounds per foot (4.05 kilonewtons per meter)

Factored bending moment: M_u = 276.4x(8.0)^2/12 = 1,464.5 foot.pounds = 1.465 foot.kips (2.0 meter.kilonewtons)

Factored shear: V_u = 274.6x8.0/4 = 549 pounds (2.47 kilonewtons)

Note: The unfactored moment (for ASD use) and shear are 1.05 foot.kips (1.42 meter.kilonewtons) and 0.4 kips (1.8 kilonewtons), respectively.

For design, we start by selecting an angle and computing the strength values. In this example, we try an angle 5-inch by 5-inch by 5-16th of an inch in thickness. The AISC provisions are used to determine the bending and shear capacity of the angle. Per requirements by AISC, the member must be at least laterally restrained at both ends. This can be provided by securing the ends to the masonry through bolts. The vertical leg of the angle at its upper portion is in compression with the maximum stress at the tow (the tip). There are no lateral horizontal loads; and the bending occurs with respect to the X-X axis of the cross-section. The section properties of this angle are

Section modulus: S_x equals 2.04 cubic inches (33,430 cubic millimeters)

Cross-sectional area: A equals 3.03 square inches (1,955 square millimeters)

Moment of inertia: I_x equals 7.44 inches to the power of 4 (3.1 times 10 to the power of 6 with a unit of millimeter to the power

of 4)

Thickness: t is 0.31 inches (8 millimeters)

Width of the angle: b is 5 inches (127 millimeters)

Bending

We first need to determine the bending capacity of the angle. The yielding bending strength is

$M_y = S_x F_y$, and the nominal flexural strength is

$M_n = 1.5M_y$ = 1.5x(2.04x36.0) = 110.2 inch.kips (12.5 meter.kilonewtons)

Lateral-torsional buckling limit state, per AISC, for a single angle under bending about a geometric axis with no lateral torsional restraints is obtained from the following equation

$M_y = 0.80F_yS_x$ = 0.80x36.0x2.04 = 58.8 inch.kips (6.6 meter.kilonewtons)

We also need to compute the critical bending moment resistance M_{cr}. For the bending moment about one of the geometric axes of an equal-leg angle with no axial compression, and no lateral-torsional restraint and maximum compression at the toe,

C_b is equal to 1.14 (lateral restraints at ends only), and

$M_{cr} = \frac{0.58Eb^4tC_b}{L_b^2}\left[\sqrt{1+0.88\left(\frac{L_bt}{b^2}\right)^2}-1\right]$

Thus

$M_{cr} = \frac{0.58\times(29{,}000)\times(5.0)^4\times0.31\times1.14}{(8.0\times12)^2}\left[\sqrt{1+0.88\left(\frac{8.0\times12\times0.31}{5.0^2}\right)^2}-1\right]$ = 205.5 inch.kips (23.1 meter.kionewtons)

The ratio M_y/M_{cr} is 58.8 divided by 205.5 and is equal to 0.286. This is less than 1.0, therefore, per AISC, the nominal capacity is computed as follows

$M_n = \left[1.92-1.17\sqrt{M_y/M_{cr}}\right]\times M_y$ but not to exceed $1.5M_y$

(Note: in this equation, M_y is 58.8 inch.kips)

$M_n = \left[1.92-1.17\sqrt{0.286}\right]\times 58.8$ = 76.1 inch.kips (6.6 meter.kilonewtons)

This value is less than $1.5M_y$ = 1.5x58.8 = 88.2 inch.kips. Thus M_n = 76.1 inch.kips (8.6 meter.kilonewtons)

Finally, we must compute the moment capacity based on the leg local buckling effect. Per AISC, since the toe of the leg is in compression, the slenderness of the leg in compression is

$\lambda = b/t$ = 5.0/(0.31) = 16.0

The limiting non-compact slenderness ratios from AISC are

$\lambda_p = 0.54\sqrt{E/F_y} = 0.54\sqrt{29,000/36.0}$ =15.3

$\lambda_r = 0.91\sqrt{E/F_y} = 0.91\sqrt{29,000/36.0}$ =25.8

Since λ is between the two limits, the leg is non-compact in flexure, and as such, we must use a reduced section modulus S_c.

Reduced section modulus: $S_c = 0.8S_x$ =0.8x2.04 = 1.63 cubic inches (26,711 cubic millimeters)

Following the AISC procedure for this situation,

$M_n = F_yS_c\left[2.43-1.72(b/t)\sqrt{F_y/E}\right]$

$M_n = 36.0 \times 1.63 \left[2.43 - 1.72 \times (16.0)\sqrt{36.0/29,000}\right]$ = 85.7 inch.kips (9.7 meter.kilonewtons)

Thus, the torsional buckling limit state (which has a smaller bending capacity) controls; and as such

M_n = 76.1 in.kips (8.6 meter.kilonewtons)

Using the LRFD approach, ϕ = 0.9, the factored capacity is

ϕM_n = 0.9x76.1 = 68.5 inch.kips = 5.7 foot.kips (7.7 meter.kilonewtons)

Strength utilization = 1.465/5.7 =0.26 is less than 1, thus design is acceptable for bending.

Using the ASD approach, parameter Ω_b is equal to 1.67 and the allowable bending capacity is

$M_n/\Omega_b = 76.1/1.67$ = 45.6 inch.kips = 3.8 foot.kips (5.1 meter.kilonewtons)

Strength utilization = 1.05/3.8 = 0.28 which is less than 1, thus design is acceptable for bending when using the ASD approach.

Shear

For shear design, using the AISC specification for angles,

k_v is 1.2 and the limiting value for the ratio h/t_w is

$1.10\sqrt{k_v E/F_y} = 1.10\sqrt{(1.2 \times 29,000)/36}$ = 34.2

In this example $h/t_w = b/t$ and is equal to 16.0, which is less than the limit. Thus with the factor c_{v2} is taken as 1.0,

$V_n = 0.6F_y b t C_{v2}$ = 0.6x36.0x5.0x0.31x1.0 = 34.0 kips (151.2 kilonewtons)

Using LRFD, ϕ_v is 0.9 and thus $\phi_v V_n$ is 0.9x34.0 = 30.6 kips (136.1 kilonewtons)

Strength utilization = 0.549/30.6 = 0.02, which is less than 1 and thus the design is acceptable for shear also.

If the ASD method is used, the parameter ψ_v per AISC is equal to 1.67. This results in an allowable shear capacity of V_n/ψ_v equal to 20.4 kips (90.7 kilonewtons).

Strength utilization = 0.4/20.4 = 0.02 which is less than 1, thus design is acceptable for shear using the ASD approach.

Deflection

The deflection is computed for the service load and is checked against the limits.

$y = \frac{(196/12)\times(8.0\times12)^4}{120\times29,000,000\times7.44}$ = 0.05 inches (1.3 millimeters), which is less than the limiting values of $L/600$ = 0.16 inches (4 millimeters) and 0.3 inches (8 millimeters). Thus the deign is acceptable for deflection.

Bearing pressure on masonry in end support areas

We also need to check the support area. The masonry compressive strength is taken as f'_m = 1,000 psi (6.9 megapascals). For a direct compression, the capacity reduction factor is taken as 0.65. With a factored load of 549 lbs applied on the masonry at the end, stress on the masonry is set equal to the compressive strength and dimension a (the length of support) is then computed, see Figure E7.6.

$\frac{V_u}{a\times b} \leq \phi(0.85f'_m)$

Solving this for a,

$a \geq \frac{V_u}{b\phi(0.85f'_m)}$ = 0.2 in (5 millimeters). Use at least 4 inches (102 millimeters) to accommodate the required edge distance for the hole needed for anchor installation.

In conclusion, the angle selected satisfies all design requirements.

Example 7.7 (Refer to Section 7.4)

Repeat Example E7.6 however; consider a situation where the interlock resistance between masonry blocks and mortar cannot be considered.

Solutions:

In this case, the masonry weight over the height of 7.5 feet (2.3 meters) is all carried by the lintel as a uniformly distributed load. Thus, we need to recompute moment, shear and deflection based on the new loads.

Service load: q = 42x7.5 = 315 pounds per linear foot

Factored load: q_u = 1.4x315 = 441 pounds per linear foot

Factored bending moment: $M_u = q_u L^2/8$ = 441x(8.0)^2/8 = 3,528 foot.pounds = 3.53 foot.kips, which is smaller the bending capacity of 5.7 foot.kips, thus design is acceptable for bending.

Factored shear =$V_u = q_u L/2$ = 441.x8.0/2 = 1,764 pounds = 1.76 kips, which is smaller than shear capacity of 30.6 kips, thus design is also acceptable for shear.

Deflection: $y = \frac{5qL^4}{384EI} = \frac{5\times(315/12)\times(8.0\times12)^4}{384\times29{,}000{,}000\times7.44}$ = 0.13 inches (3.3 millimeters), which is less than the limits of 0.16 inches (4 millimeters) and 0.3 inches (8 millimeters). Thus the beam is acceptable for deflection as well.

The support is also adequate; and as such, the angle selected is satisfactory for this new configuration.

Example 7.8 (Refer to Section 7.6)

The span length of a girt is 24 feet or 7.3 meters (which is the spacing between the two adjacent columns). The tributary width for wind (which is determined by the spacing between girts in vertical direction) is 18 feet (5.5 meters); and the total wind pressure (including the sum of pressure and suction) is 20 pounds per square foot (1.0 kilonewton per square meter). A W8 by 18 is suggested for the girt, with F_y equal to 50 kips per square inch (344.7 megapascals). Sag rods are 5-8th of an inch in diameter (16 millimeters) each with a length of 12.6 feet (3.8 meters). The W-section and sag rod configurations are similar to the details shown in Figure 7.12 of Chapter 7. Figure E7.8 shows the application of the wind load on this girt. Determine the adequacy of the W-section and sag rod designs.

Solution:

In this example, we use ASD method. The bending resistance is obtained from AISC manual.

The uniformly distributed wind load on the girt is

q_W = 20x18 feet = 360 pounds per foot (5.3 kilonewtons per meter). The applied bending moment is

$M = q_W L^2/8$ = 360x(24.0)^2/8 =25,920 foot.pounds (35.1 meter.kilonewtons)

Because the girt is attached on its compression flange to the cladding in multiple places, we can assume the compression flange is fully supported laterally. And as such, the bending resistance of this section per AISC (for ASD) is

M_R= 26.5 foot.kips (26,500 foot.pounds or 35.9 meter.kilonewtons), which is larger than the applied moment of 25,920 foot.pounds (35.1 meter.kilonewtons). Thus the W-section is adequate for the wind load pressure.

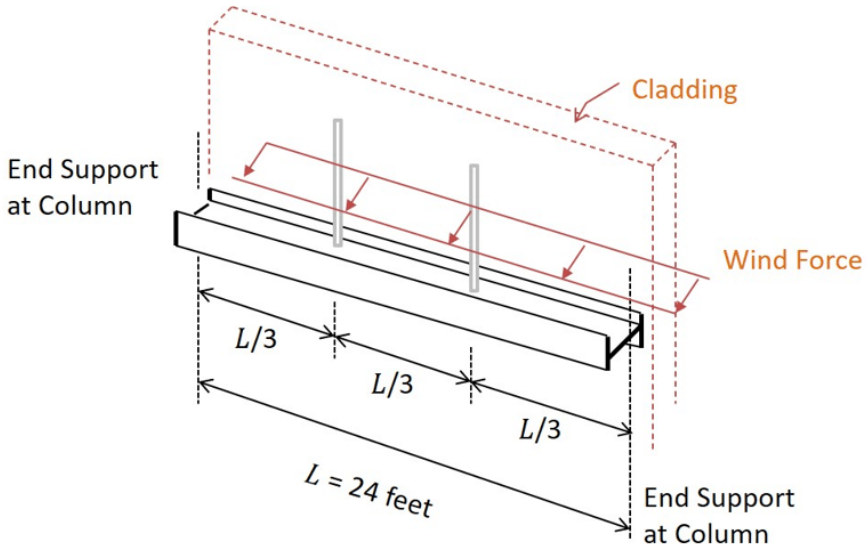


Figure E7.8 Win load application on the girt in Example 7.8

(This figure shows a beam of 24 feet in length against wind load pressure. Two vertical rods are shown spaced at $L/3$ to provide lateral support for the beam.)

To compute the net compression in the sag rod, we need to first compute the tension force developed in the sag rod as a result of carrying the weight of the girt.

Tension in a sag rod = $1.1qL/3$ = 1.1x18x24.0/3 = 158.4 pounds (0.7 kilonewtons)

In this equation, q is the self weight of the beam, which is equal to 18 pounds per foot (0.3 kilonewtons per meter).

There is a compression force developed in the sag rods, which is assumed to be equal to 1% of compression F developed in the flange of the girt. This force is obtained by dividing the bending moment by the effective depth d of the beam. Knowing that for an 8 by 18 W-section, the beam depth h is 8.14 inches and the flange thickness t_f is 0.33 inches, $d = h - t_f = 8.14 - 0.33$ = 7.81 inches (198 millimeters).

Accordingly, the compression in the flange is

$F = M/d$ = 25,920x12/7.81 = 39,826 pounds for two sag rods. Thus the compression force in any one sag rod is

$0.01F/2$ = 199.1 pounds (0.89 kilonewtons)

Net compression in a rod = 199.1 – 158.4 = 40.7 pounds (0.18 kilonewtons)

This value will need to be smaller than the buckling strength of the rod.

Allowable buckling strength of rod = $F_a A_b$, where

F_a is the allowable stress, and

A_b is the cross-sectional area. For this rod size, it is equal to 0.31 square inches.

The slenderness ratio, $k\ell/r$ for the rod is

$k\ell/r = 1.0 \times (12.6 \times 12)/0.16$ = 945

Considering the rod as a connecting element, per AISC Section J4.4, for this large slenderness ratio, the provision in Section E applies. First, we need to compute F_e

$F_e = \frac{\pi^2 E}{(k\ell/r)^2} = \frac{\pi^2 \times 29{,}000{,}000}{(945)^2}$ = 320.5 pounds per square inch (2.2 megapascals)

Then the allowable stress is

$F_e = \frac{0.877 F_e}{\Omega_c} = \frac{0.877 \times 320.5}{1.67}$ = 168.3 pounds per square inch (1.16 megapascals)

Thus,

Allowable buckling strength of a rod = $F_a A_b$ = 168.3x0.31 = 52.2 pounds (0.23 kilonewtons), which is larger than the applied load of 40.7 pounds (0.18 kilonewtons). Thus the rod design is acceptable.

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11.8: Illustrative Examples for Chapter 8

Example 8.1 (Refer to Section 8.5)

An engineer in charge of a terra cotta façade inspection suspects that 50% of terra cotta blocks have severe corroded attachment rods. The expected variation in this estimate is 0.25. Determine the sample size and the sample size error in estimating the number of openings on this façade based on 0.95 confidence.

Solution:

The confidence level of 0.95 will result in an α of 0.05 and (1- α/2) of 0.975. Thus $k_{\alpha/2}$ is 1.96. Using p = 0.50, and Δ = 0.25,

$n = \frac{1-p}{p}(\frac{k_{\alpha/2}}{\Delta})^2 = \frac{1-0.5}{0.5}(\frac{1.96}{0.25})^2$ =62

The sample error is

Error equals plus or minus $\sqrt{\frac{p(1-p)}{n}} = \sqrt{\frac{0.5(1-0.5)}{62}}$ or plus or minus 0.06 (6%).

Thus, the initial plan can start with 62 openings. As the investigation advances, the value of p can be changed and n modified.

Example 8.2 (Refer to Section 8.6)

Table E8.2 summarizes a set of 32 sample values compiled for a shaking table's accelerations for a period of 3.1 seconds. The samples were compiled at a rate of ten per second (sampling rate is 10 per second).

Table E8.2 Acceleration data

Time (sec)	Acceleration (g)	Time (sec)	Acceleration (g)
0	0	1.6	0.221
0.1	0.471	1.7	0.170
0.2	0.709	1.8	0.080
0.3	0.754	1.9	0.015
0.4	0.374	2.0	-0.061
0.5	0.361	2.1	-0.091

0.6	0.073	2.2	-0.107
0.7	-0.175	2.3	-0.053
0.8	-0.313	2.4	-0.014
0.9	-0.324	2.5	0.036
1.0	-0.324	2.6	0.060
1.1	-0.180	2.7	0.074
1.2	-0.031	2.8	0.099
1.3	0.088	2.9	0.072
1.4	0.189	3.0	0.038
1.5	0.239	3.1	0.006

a) Plot the data (g_k) in terms of time.

b) Using Microsoft Excel, obtain the absolute values of S_j for j equal to 1 to 16 and plot them.

c) Identify the dominant frequency content of the compiled data.

Solution

(a) The time story plot of this data appears in Figure E8.2(a)

(b) Using the commercial program Microsoft Excel, the S_j data are obtained and the results are plotted in Figure 8.2(b).

(c) The dominant frequency appears to be at j equal to 3. Since the sampling rate is 10, using Equation given in Section 8.6 of Chapter 8, the dominant frequency is 3 over 32 multiplied by 10, which 0.940 hertz.

Example 8.3 (Refer to Sections 7.4 and 8.6)

A simply supported beam of length l = 18 feet (5.5 meters) carries a weight of 5,000 pounds (22.2 kilonewtons). The cross section is made up of steel with (\I_x\) equal to 307 inches to the power of 4 (127.8 times 10 to the power of 6 millimeters to the power of 4) and E equal to 29,000,000 pounds per square inch (199,948 megapascals). An air conditioning unit inserts an input vibration with a frequency of 10 hertz.

Compute the fundamental natural frequency of the beam and compare it with the frequency of the input vibrations.

Solution:

From the equations given in Section 8.6, the stiffness, mass and frequency are computed as follows:

$K = \frac{48EI_x}{L^3} = \frac{48 \times 29{,}000{,}000 \times 307}{(18 \times 12)^3}$ = 42,405 pounds per inch (7,426 kilonewtons per meter)

$M = \frac{W}{g} = 5{,}000/386$ = 12,953 pounds times square per inch = (2,268 kilograms of mass)

$f = \frac{1}{2\pi}\sqrt{\frac{K}{M}} = \frac{1}{2\pi}\sqrt{\frac{42{,}405}{12.953}}$ = 9.1 hertz

The applied frequency is too close to this. Therefore, there is a possibility for resonance. To avoid resonance, we can use a stronger cross section to increase K and thus increase f to get out of phase with the frequency of the input vibrations.

Note: If we wish to have the fundamental natural period of vibration of the beam, we simply use $T = 1/f = 1/9.1$ which results in a period of 0.11 seconds.

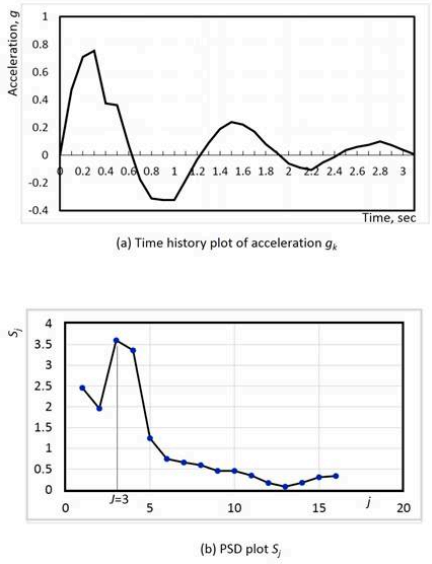


Figure E8.2 Plots for Example 8.2

(This figure shows two graphs; the upper one is a time history of a vibration over 3 seconds. The lower figure shows the frequency diagram with a peak shown at j equal to 3, which corresponds to the dominant frequency of vibration.)

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11.9: Illustrative Examples for Chapter 9

Example 9.1 (Refer to Section 9.4)

Three alternatives are offered to the owner in a façade repair case. These are

Alternative 1:

Do-nothing (this means the current cost is zero)

Yearly maintenance cost: $8,000 per year

Yearly inspection cost: $5,000 per year

Penalty: $500 per year

Alternative 2:

Partial repair now for a cost of $45,000

Yearly maintenance: $4,000 per year

Alternative 3:

Do all the required repair now for a cost of $150,000

Yearly maintenance: $1,000 per year

Saving on heating and air conditioning: $3,200 per year

Using EUAC for n = 25 years and i = 5% interest rate per year, determine which option is the best.

Solution:

Alternative 1:

P = 0

EUAC1: -5,000

EUAC2: -8,000

EUAC3: -500

Thus: Total EUAC is equal to -13,500

Alternative 2:

P = -45,000

EUAC1: -4,000

Converting P to EUAC will result in the following EUAC value

EUAC2 = $P\frac{i(1+i)^n}{(1+i)^n-1} = -45,000\frac{0.05(1+0.05)^{35}}{(1+.05)^{35}-1}$ =-3,193

Thus, the total EUAC is -7,193

Alternative 3:

P = -150,000

EUAC1: -1,000

EUAC2: +3,200

Again, converting P to EUAC results in

EUAC3 = $P\frac{i(1+i)^n}{(1+i)^n-1} = -150,000\frac{0.05(1+0.05)^{25}}{(1+.05)^{25}-1}$ = -10,643

Thus, the total EUAC is = -8,443, and the alternative with the largest total EUAC is alternative 2, with an EUAC equal to -7,193.

Example 9.2 (Refer to Section 9.4)

In Example 9.1, in Alternative 3, because of the repair work and energy efficiency, the owner is qualified for a one-time tax credit of $4,000 which will happen 2 years from now. Recompute EUAC for Alternative 3 and find out whether Alternative 3 becomes more attractive than 2.

Solution:

We first convert the future benefit F of +4,000 to an equivalent present worth, P using n equal to 2 years. Then we find the EUAC for this value.

$P = \frac{F}{(1+n)^n} = +4,000\frac{1}{(1+.05)^2}$ =+3,628

EUAC3 = $P\frac{i(1+i)^n}{(1+i)^n-1} = +3,845\frac{0.05(1+0.05)^{25}}{(1+.05)^{25}-1}$ = +257

The new total EUAC for Alternative 3 = - 8,186; and Alternative 2 is still more attractive.

Example 9.3 (Refer to Section 9.4)

In a façade repair case, the owner has only two alternatives, either do nothing or repair now. Here is the information for these alternatives.

Alternative 1:

Do nothing (this means the current cost is zero)

Yearly maintenance cost: $2,000 in the first year; but this will increase by $500 every year.

Yearly inspection cost: $1,800 per year

Penalty for not inspecting: $600 per year

Alternative 2:

Do all the required repair now for a cost of $200,000

Yearly maintenance: $1,000 per year

Using the EUAC method for an invest period of 30 years with an annual interest rate of 4%, determine which option is preferred.

Solution:

Alternative 1:

This alternative involves the gradient series with G equal to 500. An equivalent End-of-period payment A_{eq} the gradient series is

$A_{eq} = \frac{G}{i} - \frac{nG}{i}[\frac{i}{(1+i)^n-1}] = \frac{500}{0.04} - \frac{30\times500}{.04}[\frac{0.04}{(1+0.04)^{30}-1}]$ =-5,814

Note that this is a negative value, thus

(EAUC)*eq* = $A + A_{eq}$ = -(2,000+5,814) = -7,814

Thus, the total EUAC is -(7,814+1,800 +600) = -10,214

Alternative 2:

P = -200,000

EUAC1: -1,000

Converting P to EUAC results in

EUAC2 = $P\frac{i(1+i)^n}{(1+i)^n-1} = -200,000\frac{0.04(1+0.04)^{30}}{(1+.04)^{30}-1}$ = -11,566

Thus, the total EUAC for this alternative i -12,566; and as such, Alternative 1 becomes more attractive (as long as there will not be any safety issues involved).

Example 9.4 (Refer to Section 9.4)

Repeat Example 9.3 but use the Net Present Worth (NPW) method.

Solution:

For Alternative 1, we use the total EUAC to find NPW (in other words, we use the equation to convert A to P).

NPW1 = $P = A\frac{(1+i)^n-1}{i(1+i)^n} = -10,214\frac{(1+0.04)^{30}-1}{0.04(1+0.04)^{30}}$ = -176,621

And for Alternative 2, We first convert the annual cost A to P, then add it to the current cost of 200,000.

NPW2 = $P = A\frac{(1+i)^n-1}{i(1+i)^n+} + P_1 = -[1,000\frac{(1+0.04)^{30}-1}{0.04(1+0.04)^{30}} + 200,000]$ = -217,292

Thus Alternative 1 with a larger NPW is preferred, which confirms the finding in Example 9.3.

Example 9.5 (Refer to Section 9.5)

Windows on the exterior façades of three-story condominium cost 3.2 million dollars to replace. The useful life of windows is estimated as 20 years and they were installed 5 years ago. Conduct a component (straight-line) funding analysis to list yearly investment needed for timely replacement of the windows. Use 3.4% inflation rate. The association will invest the distribution in a saving account resulting an interest rate of 1.5% per year. Currently, there are no funds allocated to window replacement. The estimated cost for minor repair, painting and cleaning is $20,000 in year one and increases by 3% every year.

Solution:

This analysis is part of a comprehensive reserve study and intended to demonstrate the computation procedure conducted for one component and yearly distribution needed to accumulate funds. It is noted that generally any of the many available software for reserve studies is used. In order to get an estimate of yearly distribution needed, we first compute consider the present worth

as P = 3.2 million dollars and convert it to the annual cost A using i equal to 0.034 and n equal to 15 years.

$A = P\frac{i(1+i)^n}{(1+i)^n-1} = 3,200,000\frac{0.034(1+0.034)^{15}}{(1+.034)^{15}-1}$ = 275,868

The future cost of replacement is

$F = P(1+i)^n = 3.2 \times (1+0.34)^{15}$ = 5.28 million dollars.

The association can start setting aside $275,868 in year one considering that it will increase by 3% every year (per recommendations by industry standards). Using a spreadsheet, we can prepare a table summarizing the distribution, interest received at 1.5% per year and expenses to determine the funds left at the end of each year. Also, it is noted that in year 15, there will be a major cost for replacement of 5.28 million dollars. In the first year, the interest received is based on the initial distribution ($275,868). In subsequent years, the interest will be based on the available funds in the start of the year. For example, $260,006 in the year 2 and $527,350 in year 3 and so on.

The following table summarizes the amount accumulated at the end of each year. Assuming there are no expenses per year using the spreadsheet.

Amount accumulated at the end of each year

Year	Beginning Fund	Distribution	Expenses	Investment
1	0	275,868	20,000	4
2	260,006	284,144	20,600	3
3	527,450	292,668	21,218	1
4	806,812	301,448	21,855	1
5	1,098,508	310,492	22,510	1
6	1,402,968	319,807	23,185	2
7	1,720,633	329,401	23,881	2
8	2,051,963	339,283	24,597	3
9	2,397,427	349,461	25,335	3
10	2,757,515	359,945	26,095	4
11	3,132,727	370,744	26,878	4
12	3,523,583	381,866	27,685	5
13	3,930,618	393,322	28,515	5
14	4,354,384	405,121	29,371	6
15	4,795,451	417,275	5,280,000	7

As seen in the table, after all expenses, the association will meet the funding requirement with a small surplus of $4,657.

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11.10: Illustrative Examples for Chapter 10

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12: APPENDIX B - ADDITIONAL OPEN-ACCESS (OA) MATERIALS

Appendix Synopsis

This appendix presents a curated selection of resources for further study on the rehabilitation and repair of building envelopes. The references are primarily technical in scope and are intended to complement and extend the material discussed in this volume, enabling readers to explore specific topics in greater depth.

In the selection process, priority was given to sources that maintain technical rigor and objectivity. Materials with overt commercial content or promotional intent have been deliberately excluded in order to preserve the academic integrity and neutrality of the recommended references.

Introductory Remarks

Given the open educational resource (OER) nature of this book, it is anticipated that users may wish to contribute additional references to this appendix as new materials become available. To ensure that such contributions maintain the quality, accessibility, and integrity of the resource, the following guidelines are recommended:

- Updating and removal of outdated sources – It is recognized that some listed materials may, over time, be removed from their original platforms, updated, or become otherwise inaccessible. Users are encouraged to provide comments indicating when a source is no longer available or when a more current version exists.
- Avoidance of commercial bias – To preserve the educational and non-commercial intent of this OER, contributors should refrain from adding materials that promote specific products or display overt commercial bias.
- Open access requirement – Only resources that are freely available to all users, without cost or purchase obligations, may be added to this appendix.
- Contextual description – When submitting a new reference, users are encouraged to provide a brief description of its subject matter and to indicate the corresponding section of the book to which it relates.

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12.1: Open-Access Materials Recommended for Chapter 1
12.2: Open-Access Materials Recommended for Chapter 2
12.3: Open-Access Materials Recommended for Chapter 3
12.4: Open-Access Materials Recommended for Chapter 4
12.5: Open-Access Materials Recommended for Chapter 5
12.6: Open-Access Materials Recommended for Chapter 6
12.7: Open-Access Materials Recommended for Chapter 7
12.8: Open-Access Materials Recommended for Chapter 8
12.9: Open-Access Materials Recommended for Chapter 9
12.10: Open-Access Materials Suggested for Chapter 10

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12.1: Open-Access Materials Recommended for Chapter 1

(1) Pertinent section: 1.7

Link to OA material: (https://youtu.be/rAVKjvweP00 [youtu.be])

Platform: YouTube

Description: Common terminologies in façade inspections are covered in this video

Date (year): 2025

Comments: None

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12.3: Open-Access Materials Recommended for Chapter 3

(1) Pertinent sections: 3.1 and 3.2

Link to OA material: [youtu.be]

Platform: YouTube

Description: Bearing walls made up of brick and CMU blocks

Date (year): 2025

Comments: This twenty-six-minute lecture video covers brick and CMU nearing walls, which also at times act as façades. Their details, reinforcement design and damage issues are explained.

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12.6: Open-Access Materials Recommended for Chapter 6

(1) Pertinent section: 6.3

Link to OA material: Snow drift computation interactive program

Platform: Web

Description: Snow drift load values and profile

Date (year): 2025

Comments: This interactive program computes snow drift loading on roofs based on information on the roof characteristics and ground snow load value. However, it is noted that the calculations are based on the old ASCE provisions. There have been changes implemented in ASCE-22 that are not reflected in the methods used in this source.

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12.7: Open-Access Materials Recommended for Chapter 7

(1) Pertinent section: 7.3

Link to OA material: [youtu.be]

Platform: YouTube

Description: Shelf angle design

Date (year): 2025

Comments: This 22-minute video gives a lecture on shelf angle sizing and design using PowerPoint slides.

(2) Pertinent section: 7.4

Link to OA material: [youtu.be]

Platform: YouTube

Description: Design of angles used as lintels

Date (year): 2025

Comments: This 19-minute video gives a lecture on lintel design and sizing and design using PowerPoint slides.

(3) Pertinent sections: 7.3 and 7.4

Link to OA material: Angle_Bending_Strength

Platform: Excel worksheet

Description: This spreadsheet calculates the bending moment capacity of an angle used as a lintel or in shelf angles that are connected to structural steel. The link takes you to the spreadsheet in an open Google Sheets.

Comments: Using the cross-section properties of an angle and its span length, the spreadsheet computes the nominal bending strength using the American Institute of Construction (AISC) methods.

(4) Pertinent sections: 7.3.1

Link to OA material: [youtu.be]

Platform: YouTube

Description: Design of anchors in concrete

Comments: This one-hour long lecture video describes design of anchors in concrete following the procedures of the American Concrete Institute (ACI).

(5) Pertinent sections: 7.3.2

Link to OA material: [youtu.be]

Platform: YouTube

Description: Design of anchors in masonry

Comments: This twenty-minutes long lecture video describes design of anchors in masonry following the procedures of the

American Concrete Institute (ACI) and other pertinent codes.

(6) Section 7.6

Link to OA material: [youtu.be]

Platform: YouTube

Description: Design of a girt in façades

Comments: This twenty-one-minutes long lecture video describes design requirements for a girt and its sag rods with an example problem.

(7) Section 7.7, Exercise Number E7.3

Link to OA material: [youtu.be]

Platform: YouTube

Description: Design of anchors in masonry

Comments: This twenty-three-minutes long lecture video describes a simple method for computing the cross-sectional properties of built-up sections.

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12.8: Open-Access Materials Recommended for Chapter 8

(1) Pertinent section: 8.6

Link to OA material: [youtu.be]

Platform: YouTube

Description: Computation of the fundamental natural frequency of vibration in beams.

Date (year): 2025

Comments: This short lecture video describes the procedure in computing the fundamental natural frequency of simply-supported and cantilever beams based on their stiffness and mass. Simple equations for a variety of beam conditions and loadings are provided.

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12.9: Open-Access Materials Recommended for Chapter 9

(1) Pertinent section: 9.1

Link to OA material: Engineering economics calculator

Platform: Web

Description: Engineering economics computations

Date (year): 2025

Comments: The interactive calculator allows converting F, A, G and F values to one another based on the interest rate and the period of investment.

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12.10: Open-Access Materials Suggested for Chapter 10

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Date (year):

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GLOSSARY

A

Afar examination

|

Inspection conducted using binoculars.

Anchor

|

Bolt and nut system used to attach shelf angles or brackets to masonry.

Annual examination

|

An inspection conducted every year.

Awning

|

A covering attached to the exterior wall of a building, typically above windows, doors, or over patios and decks.

B

Backer rod

|

A flexible rope-like material used as fillers between joints.

Bond beam | A beam made up reinforced concrete, or reinforced masonry blocks, that is built on masonry walls to provide support for floor joists

Brick and CMU positions | Brick and CMU positions are referred to as stretcher, soldier, header, stretcher rowlock, sailor and rowlock.

Building corner | For façade inspection purposes, it is at the intersection of two walls.

C

Canopy | A projecting overhead covering that provides protection, often built over entrances, windows, or walkways.

Categories I | These vary across municipalities and generally refer to the building types in terms of the occupancy (for façade inspection and safety purposes).

Caulk | A low- or intermediate-performance compound used as a sealer mainly in interior applications. Note: see definition for “sealant,” which is a high-performance compound for exterior façade applications.

Change order | A request to change a detail that is different from the one on the original drawing.

Cladding | The material and/or element used as the finish on the exterior of the façade.

Close examination | An inspection that requires an architect or engineer to use scaffolds and conduct a close evaluation of the façade condition.

CMU | Concrete masonry unit.

Control joint | A cut made into concrete slabs to allow expansion and contraction in concrete – Note: this is different from an expansion joint.

Coping stone | A piece of stone at the top of a parapet wall. The types are plain straight, twice weathered and once weathered.

Cornice | A decorative molding or projection that typically runs along the top of a building constructed just below the roofline.

Course of brick | One layer of brick in horizontal direction.

Crack meter | A device used to monitor the width and growth of cracks in masonry. Simple ones are made up of two pieces with markings. Each piece is mounted on one side of the crack. The two pieces slide over each other providing information on crack growth.

Critical examination | A façade examination that requires close observation of wall condition using scaffolds and covering the required façade areas per recommendations by the municipality.

D

Door terminologies | Terminologies pertinent to doors are jamb (an upright of frame on which door is hanging; rabbet (groove or recess at the edge); and threshold. Doors are often marked with ratings as one-hour, two-hour, etc., which refer to fire protection.

Drone | An unmanned aircraft system (UAS) designed to capture detailed visual data of a building's exterior, including hard-to-reach areas, to assess the envelope integrity.

Drop | A vertical area of façade designated for inspection when using moveable scaffolds.

Due diligence inspection | An inspection conducted before sale of the property.

E

Efflorescence | A crystalline deposit on surfaces of masonry walls.

EIFS | External Insulation Finish System, also known as synthetic stucco.

Envelope | A term used to generally refer to the building enclosure including the roof.

Expansion joint | A vertical gap in walls to allow wall expansion and contraction – note: this is different from a control joint.

F

Façade | A term used to generally refer to the building enclosure excluding the roof.

Façade inspection | Evaluation of the building enclosure for safety purposes as mandated by a local jurisdiction.

Flashing | A metal or neoprene sheets used to wash water out at areas above windows and doors, under the coping stone, and between wall and foundation.

G

Girt | A horizontal beam, usually installed in the mid-height between two floor levels, to help reduce the bending of the cladding in the vertical direction and protect it from damage due to wind pressure.

Green roof | A roof with vegetation grown on it.

Green wall | An energy-saving wall with vegetation grown on it.

I

Inspection | See façade inspection.

L

Ledger beam | A timber beam, usually with a nominal thickness of 2 inches (51 mm) secured to the wall via anchor bolts to provide supports for hangers used in timber joists.

LIDAR | Laser imaging, detection and ranging is a laser-based method to identify surface shape of walls and other building features.

Lintel | A beam at the top of the window or door frame or other openings.

M

Maintenance | Periodic inspections to make sure the building envelope is safe. This may or may not involve repair.

Mortar | A cement-based material used in between masonry blocks.

N

NDE method | A non-destructive, non-invasive evaluation method.

NDT method | A non-destructive, non-invasive test method.

O

Opening | A façade element (such as a brick) removed for inspection of the condition behind the wall.

OSHA | Occupational Safety and Health Administration.

P

Parapet wall | A short wall extended above the roof.

Porch | An open structure made up of posts and beams attached to the main building, usually on the back. Porches are mainly made up of wood or steel and contain external stairs and decks.

Professional of the record | An engineer or architect in charge of façade examination.

R

Reconstruction | A portion or entire building envelope rebuilt to its original condition.

Rehabilitation | A series of major replacements or changes in a portion or the entire envelope to maintain the intended function.

Render | See stucco.

Repair | A series of minor replacements or changes in a portion or the entire envelope to maintain the intended function.

Reserve study | An engineering and financial analysis conducted to determine the capital investment and funds needed to set aside for the repair, rehabilitation, and replacement of major components in multi-occupancy buildings. This study is often required by certain jurisdictions to ensure that adequate reserves are maintained for future maintenance and capital expenditures.

Retrofit | A series of changes that would improve the function of a portion or entire envelope to bring it to a higher level to meet new demands.

S

Scaffold | A moveable or stationary temporary structure used for façade repair.

Sealant | A compound (often high-performance) used for water proofing in exterior façade applications. Note: see the definition for “caulk,” which is a lower-performance compound usually for interior applications.

Shelf angle | An angle attached to the structure to support a veneer wall.

Spandrel beam | A beam that runs around the edge of each floor and is integrated with the floor slab.

Stairs | Terminologies used for stairs are “going,” “nosing,” “rise,” and “tread.” A landing is a small area when the stair turns.

Steel relief angle | Same as shelf angle.

Stucco | A plaster-like exterior finish applied in multiple layers over a substrate such as metal lath, wire mesh, or masonry in façades.

Swing stage | An area where moveable scaffold is installed for façade inspection.

T

Terra-cotta block | A plain or ornamental block made up of baked clay attached to the structure through metal anchors.

Thermal break | A component or material with low thermal conductivity used to reduce heat transfer and improve energy efficiency.

Tilt-up panels | Reinforced concrete panels cast horizontally and then lifted into place to form the building façade.

Tuckpointing | A repair done on mortar between masonry units.

U

UAS | Unmanned aircraft system. Also. see the definition for “drone.”

W

Wall | Different types of walls are curtain, load bearing, parapet (a wall about 42 inches tall extending above the roof, veneer, tilt-up, and green walls.

Water test | A test by splashing water over façade usually for about 30 minutes or so to detect leaks.

Weep hole | A small opening, usually at the mortar joint, to allow moisture out.

Wick | A rope used in weep holes to drain condensation water and moisture out.

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