Conceptual Seismic Design Guidance for New Reinforced Concrete Framed Infill Buildings

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Table of Contents

List of Figures

1 Introduction

1.1 Preface

Throughout the world, reinforced concrete frame buildings with masonry infill walls house families, shelter school children, and provide offices for workers. These buildings are functional, durable, and economical. All too often, though, these buildings perform poorly in earthquakes. Some collapse and kill the people inside, and many are badly damaged, requiring demolition or expensive repairs. Sometimes, poor construction quality or a lack of engineering design is at fault. In many cases, though, the engineering design itself is to blame.

Despite the stiffness and strength infill walls possess, building codes around the world lack guidance on modeling and designing infill walls as structural elements, and many engineers have been taught not to consider them as such. Engineers therefore often ignore infill walls during structural design or presume that they will have only beneficial effects. This simple yet fundamental oversight often dooms buildings to poor earthquake performance. For example, many multi-story reinforced concrete buildings with masonry infill walls collapsed at the ground level from the 1999 Chi-Chi, Taiwan earthquake (Figure 1). These buildings typically had commercial space or parking at the ground floor and infill walls in the stories above.

Figure 1. Collapsed concrete frame buildings in the 1999 Chi-Chi, Taiwan earthquake (Photo credits: Jack Moehle,left, and Stephen Mahin,right; courtesy of the National Information Service for Earthquake Engineering, EERC, University of California, Berkeley)

This kind of destruction does not have to happen; infill buildings can survive strong shaking without collapse. Despite the challenges, engineers can design new seismically robust yet economical frames with infill walls. The key is to work *with* infill walls by ***always*** considering them as structural elements during the design process, and by including them in the lateral system when appropriate. Throughout this document, buildings with infill walls are referred to as “infill buildings” to emphasize the structural importance of infill walls. Due to the interaction between infill walls and the frame, infill buildings can exhibit complex seismic behavior, some of which remains poorly understood despite significant research. Recent earthquakes in Italy and Mexico have shown that some design approaches intended to improve the seismic behavior of infill buildings have not worked as well as expected. This document incorporates these lessons where possible. The authors recognize that post-earthquake investigations and research studies are still needed in order to improve the design and construction of infill buildings.

The purpose of this document is to present five strategies for designing new infill buildings while considering the infill walls as structural elements. However, framed infill structural systems are not suitable for every building, and in considering these strategies, designers might determine that different structural systems are more appropriate for their buildings. This document also describes potential problems with infill buildings that lead to poor seismic performance and explains typical infill building configurations and uses. Some of the design strategies included here could be applicable for seismic strengthening schemes, but their use in retrofitting existing buildings is outside the scope of this guide.

This document is not intended as a building code. It is a guide that is meant to supplement, but never supersede, the codes and regulations in force in the jurisdiction of the building under design. In drafting this document, the authors have drawn from codes and practices around the world. The intent of the guide is to demonstrate concepts for considering infill walls as structural elements and not to provide all the details necessary for designing buildings with infill.

As previously noted, **this document refers to any reinforced concrete frame building with infill as an "infill building." By contrast, reinforced concrete frame buildings with infill that has been *intentionally designed* as an integral component of the earthquake-resisting system are referred to as "framed infill buildings."**

The intended users of this guide are engineers who design buildings in countries where infill construction is common. Users should have a basic knowledge of fundamental earthquake engineering principles and should be familiar with equivalent lateral force procedures.

1.2 Potential Problems in Infill Buildings that Lead to Poor Seismic Performance

Modeling Infill Buildings as Bare Concrete Frames

Many researchers and practitioners in the earthquake engineering community recognize that it is problematic to ignore the effects of infill on the seismic performance of buildings. The World Housing Encyclopedia reinforced concrete frame tutorial (Murty et al., 2006), for example, provides the following guidance:

> *Masonry infill walls should not be used UNLESS they are specifically designed by an engineer to:*
>
> - *Work in conjunction with the frame to resist the lateral loads, or*
> - *Remain isolated from the frame.*

However, many engineers around the world do not heed this advice. Instead, they model concrete frames with non-isolated infill panels as bare frames, thereby ignoring the infill, with the exception of two indirect methods that account for limited aspects of the infill's contribution. These indirect methods are 1) include the infill walls' mass in the seismic weight calculations, and 2) use a code-prescribed empirical formula for infill buildings for the natural period of vibration (see Kaushik et al. 2006 for a comparison of code empirical formulae). Some engineers do not include infill walls in their seismic calculations because they fear that building occupants will later remove infill walls during remodeling and make the original calculations incorrect. These engineers believe that excluding the lateral strength of infill walls from the original design is conservative because they assume that infill walls add strength.

While it is true that occupants may remove infill walls, and that infill walls can add strength in some locations where they are placed, this does not remove the designer's responsibility to investigate the response of the building with and without its infill walls. There are logical ways to consider the effects of infill walls being removed or relocated by users, rather than simply ignoring them in the design process. For example, Eurocode 8 advises designers to disregard every third or fourth panel in a planar frame, as a way to account for potential changes to the infill panel arrangement made later by occupants (Eurocode 8, section 4.3.6.3.1).

For the purposes of this document, masonry walls within a concrete frame building are considered infill walls if they are located within the frame or if they otherwise interact structurally with the frame despite being located outside the frame lines. Walls that are not within the frame and that do not interact with it are considered partitions. Most infill buildings also have partitions, which are typically slender and can fail out-of-plane, creating a hazard for people nearby. Seismic protection of partitions is an important consideration in the overall earthquake safety of infill buildings, but this document is not intended to provide detailed guidance on partition protection techniques. Some of the approaches for the out-of-plane support of infill walls isolated from the frame (see Design Strategy D later in this document) could be applied to partitions.

Vertical and Plan Irregularities Lead to Concentrated Deformations

When engineers do not treat infill walls as structural elements, they tend to miss the configuration problems that infill walls can create. These configuration problems concentrate forces and deformations in a relatively small number of the building's reinforced concrete columns, rather than distributing forces and deformations throughout the frame. These additional demands can be too much for even well-detailed columns to withstand and often prove disastrous for weak, poorly-detailed columns. The three most potentially damaging configuration problems caused by irregular distributions of infill panels are weak and / or soft stories, torsion, and captive columns. This section summarizes each problem.

Weak and / or Soft Stories

The most common configuration problem is a weak and/or soft story (typically at the ground level), created by the absence of infill walls or the presence of many fewer infill walls than the story above and/or below. Weak and/or soft stories are often created in the process of satisfying the functional requirements of street-front or ground-level uses, such as shops or parking.[1] During an earthquake, the deformations concentrate in the weak and/or soft story, as Figure 2 shows. Reinforced concrete columns in soft and weak stories can experience lateral deformations that are more than 10 times those in the stories with substantial infill walls (Miranda, 2014).

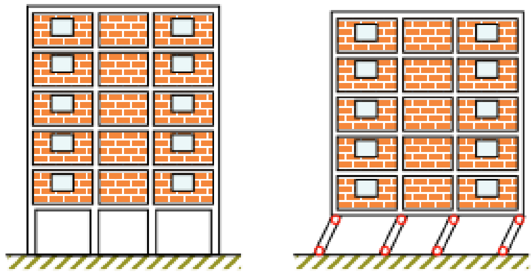


Figure 2. Concentrated deformations in soft and weak open ground story, leading to sidesway collapse of the ground story (Credit: Eduardo Miranda, Stanford University)

In many cases, deformations in the weak and/or soft story exceed the columns' deformation capacity, and the story collapses. Such collapses have occurred in many recent earthquakes, including 1999 Chi-Chi, Taiwan; 1999 Kocaeli, Turkey; 2001 Gujarat, India; 2003 Bourmedes, Algeria; 2005 Kashmir-Kohistan; 2006 Yogyakarta, Indonesia; 2008 Wenchuan, China; and 2009 Padang, Indonesia (Figure 3) and 2009 L'Aquila, Italy (Figure 4).

Figure 3. Soft-story collapse of infill buildings from the 2009 Padang, Indonesia earthquake (Photo credits: Tim Hart, Lawrence Berkeley National Laboratory)

[1] ASCE 41-13 defines a soft story as one whose stiffness is less than 70% of the lateral-force-resisting system stiffness in an adjacent story above, or less than 80% of the average lateral-force-resisting system stiffness of the three stories above. ASCE 41-13 defines a weak story as one where the sum of the shear strengths of the lateral-force-resisting system in any story in each direction is less than 80% of the strength in the adjacent story above.

Figure 4. Story collapse due to infill failure, 2009 L'Aquila, Italy earthquake (Photo credits: K. Mosalam)

Even if buildings have uniform infill wall configurations over their entire heights, unreinforced masonry infill walls are prone to experience early brittle failure, often cracking and falling out of plane. Infill wall failures, if concentrated at one story, can lead to the formation of a soft and weak story and create the deformation concentration problems previously discussed. Figure 5 shows how infill panel failures can create a weak story.

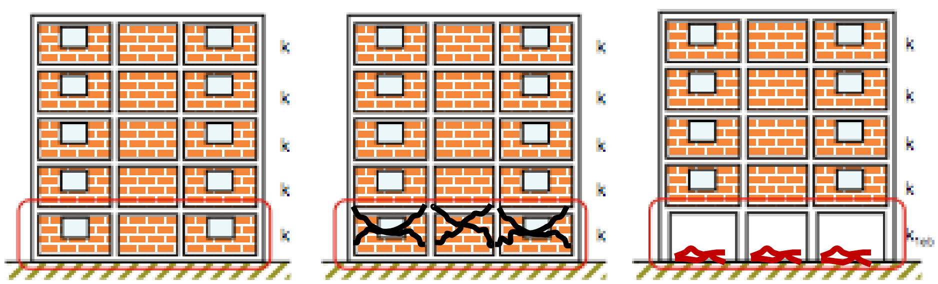


Figure 5. Weak and soft story created by infill panel failures (Credit: Eduardo Miranda, Stanford University)

Torsion Created by Plan Irregularities

A non-uniform spatial distribution of infill walls in the building plan—usually for functional reasons such as placing windows and open commercial spaces on the street frontage, but full walls adjacent to neighboring buildings —can create torsion during seismic loading. The building's torsional response places additional deformation demands on columns located in the more flexible portions of the building, and may cause them to fail. Torsion is particularly common in buildings that occupy a street corner and have an open ground floor for shops. In such buildings, the ground story columns on the street frontage can experience substantial additional deformation demands.

Captive Columns

Architectural or functional requirements sometimes call for large openings, such as for windows, in infill walls. If an opening is adjacent to a column, the partial-height infill can restrain the portion of the column below the opening, leading to a "short," or "captive," column effect. Under seismic loading, the short columns are forced to deflect the same amount as the full-height columns in the same plane of lateral resistance, leading to larger shear forces in the short columns than was anticipated during design.

This can result in a brittle shear mode of failure in columns, instead of the ductile flexural failure mode that the designer likely intended.

School buildings are especially vulnerable to short column effects because of the functional need for natural lighting and ventilation in classrooms. Schools often have exterior walls with significant portions of partial-height infill and large numbers of captive columns. The photos in Figure 6 show damage in schools from the 2009 Padang, Indonesia earthquake caused by the short-column effect.

Figure 6: Schools with shear damage caused by the short-column effect following the 2009 Padang, Indonesia earthquake (Photo credits: Tim Hart, Lawrence Berkeley National Laboratory)

The type and strength of the masonry used in the infill panel, and panel strength and stiffness relative to frame strength and stiffness, significantly impact the effects of infill on the behavior of the building. For example, hollow clay tile infill panels are much less likely than strong double wythe brick panels to create captive columns in a strong concrete frame. In a very weak frame, however, hollow clay tile might be stiff and strong enough to create captive column damage.

Interaction between Infill Panels and Frame

A reinforced concrete frame will deform in a flexural mode during seismic loading, while infill panel deformation is dominated by shear. This difference in the deformation pattern causes the infill wall to resist the frame deformation through diagonal compression, which in turn results in forces applied along the contact surface between the frame and infill (as Figure 7 shows).

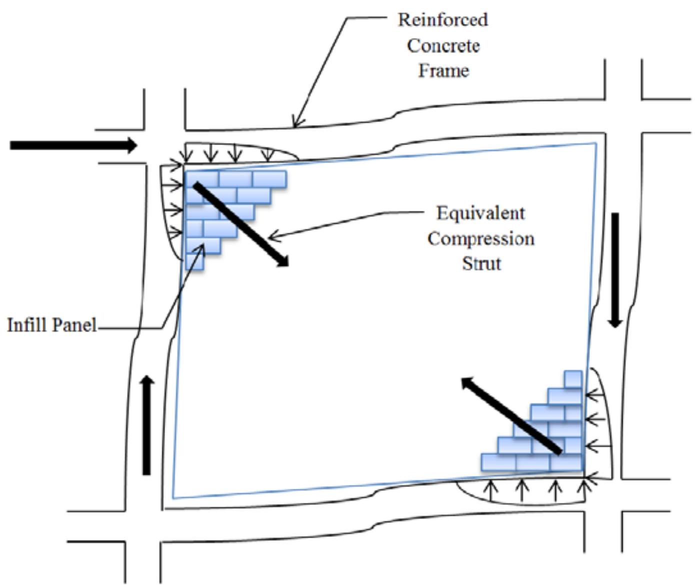


Figure 7. Infill walls resist frame deformation through diagonal compression.

For strong infill, the forces transferred to frames can be high and can introduce additional shear demands into the columns. The additional demands can be large enough to cause shear failure of the adjacent columns. Figure 8 shows a case where infill walls have induced shear failures in adjacent columns and beam-column joints.

Figure 8. Column shear damage caused by interaction between frame and infill during the 1988 Udaypur, Nepal earthquake. (Photo credit: World Housing Encyclopedia, Report 145)

Other Problems

Other design or construction characteristics that can contribute to poor seismic performance of infill buildings include:

- Poor reinforced concrete detailing or a lack of seismic detailing

- Buildings that step down slopes
- Incremental addition of floors
- Changes of use, such as residential buildings being used as schools, which can introduce forces that the buildings were not designed to resist
- Internal modifications, such as removing infill walls
- Poor detailing or construction of the joints between infill panels and concrete frames
- Cavity walls without proper ties across the air gap between the wythes
- Slender masonry partition walls (i.e., those not within frame lines) that can fail out-of-plane
- Large setbacks over the height of buildings

Further examples of earthquake damage can be found on the Framed Infill Network website (www.framedinfill.org).

1.3 Typical Infill Building Configurations and Uses

Reinforced concrete (RC) frame construction with masonry infill walls is practiced extensively in many parts of the world with high levels of earthquake hazard, including many parts of Asia, Central and South America, and the Mediterranean. Frames with infill comprise more than 50% of the building stock in major urban centers in Turkey and about 30% of the entire housing stock in Greece. In Nepal, frames with infill are considered one of the most popular and emerging building types (World Housing Encyclopedia, 2013). Recent earthquakes in India, Pakistan, China, Algeria, Taiwan, Italy, Haiti, and Indonesia have caused damage to reinforced concrete frames with infill, underscoring the prevalence of these buildings around the world. Figure 9 through Figure 16 show examples of these typical buildings.

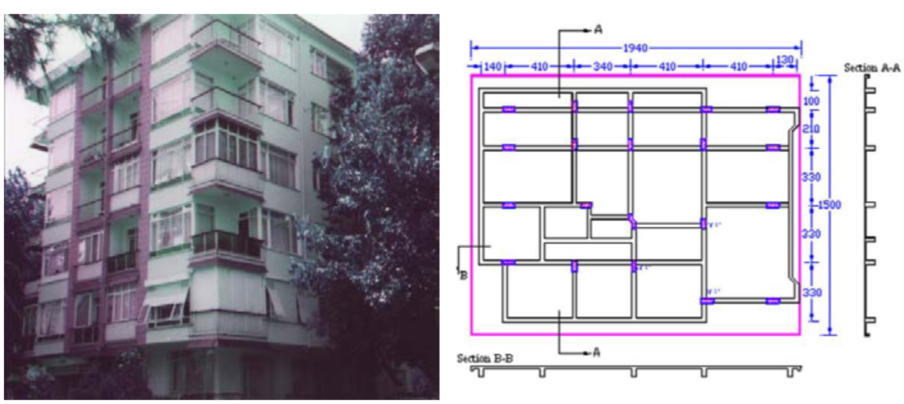


Figure 9.Multi-family residential buildings in Turkey (Photo credit: World Housing Encyclopedia, 2013)

Figure 10. Multi-family residential buildings in Italy (Photo credit: World Housing Encyclopedia, 2013)

Figure 11. Residential buildings with hollow clay tile masonry infill under construction in Algeria (Photo credit: S. Brzev, World Housing Encyclopedia, 2013).

Figure 12. Mixed use and multi-family residential mid-rise buildings in Karachi, Pakistan. (Photo credit: Greg Deierlein, Stanford University)

Figure 13. Hospital in Kathmandu, Nepal (Photo credit: Hari Kumar, GeoHazards International

Figure 14. Residential low-rise (left) and high-rise (right) buildings in Kathmandu, Nepal. (Photo credits: Janise Rodgers, GeoHazards International)

Figure 15. Schools in Fort Liberté (left) and Cap-Haitien (right), Haiti. (Photo credits: Justin Moresco, GeoHazards International)

Figure 16. Commercial/office low-rise buildings in Delhi, India (left) (Photo credit: Tom Tobin, GeoHazards International); Commercial/office low-rise buildings in Dehradun, India (right) (Photo credit: Janise Rodgers, GeoHazards International)

Typical building occupancy types and applications are summarized in Table 1, and different types of masonry infill wall material are described in Table 2.

Table 1. Typical building types and features

Building occupancy type	**Typical number of stories**	**Additional observations**
Residential with parking or shops at ground story	Low rise: 2 to 4 stories	• Weak story conditions are common due to parking or shops at ground story • Building footprints typically rectangular with aspect ratios between 1:1 and 3:1 • Geometric irregularities abound: irregular column arrangements, incomplete frames, overhangs, etc.
	Mid rise: 5 to 10 stories	• Weak story conditions are common due to shops or parking at ground story • Building footprints typically rectangular with aspect ratios between 1:1 and 2:1, with 3:1 + in some areas • Geometric irregularities abound: irregular column arrangements, incomplete frames, overhangs, etc. • Thin and short (in plan) reinforced concrete walls around lift are common
	High rise: 10 to 20 stories	• Parking at ground story rather than shops • Columns and framing usually regular but re-entrant corners common
Residential with residences at ground story	Low rise: 2 to 4 stories	Usually multiple units per building
Commercial/Office	Great variety in number of stories	• Great variety in floor plans • Lower wall density than residential • Low rise buildings often smaller • Prone to same irregularities as residential buildings
School	1 to 4 stories	• Large aspect ratios in plan • L, U, T shaped plans common • Large windows on one side and partial-height infill walls create torsion and captive column problems
Hospital	Low rise: 1 to 3 stories	Great variety in floor plans
	Mid rise: 4 to 8 stories	Great variety in floor plans

In urban areas, infill buildings are often built next to other buildings and have solid infill walls along the property line. The distribution of these large, solid infill walls in plan affects the building's seismic response. Figure 17 shows several common plan distributions of walls with solid infill panels for residential and commercial buildings, and for schools, which often fundamentally differ from other infill buildings in configuration and height. Interior configurations of infill panels vary widely in residential and commercial buildings and depend on architectural and functional considerations. Both interior and exterior panels can have openings for doors and windows, and penetrations for ventilation fans and building utilities. Openings significantly affect panel behavior. Infill buildings also commonly have interior unreinforced masonry partitions that are not within frames and do not typically interact structurally with the frame. Masonry walls that interact with the frame are considered infill walls for purposes of this document. Unreinforced masonry partitions are typically slender and thus susceptible to out-of-plane failure.

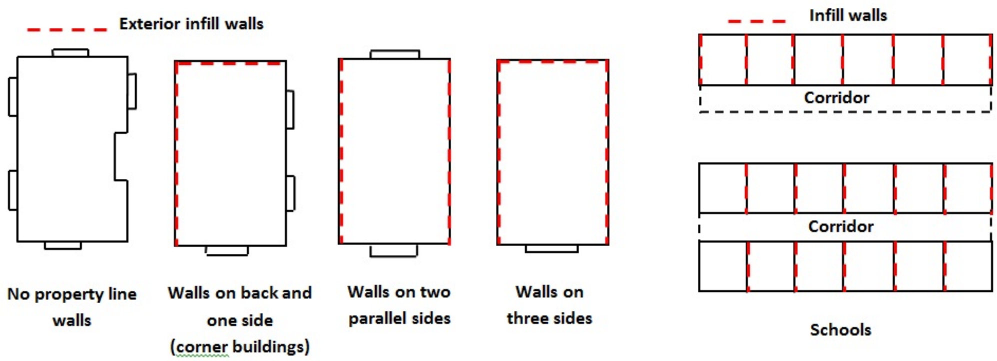


Figure 17. Typical distributions of solid infill panels in residential and commercial buildings (left); schools (right). Interior infill panel configurations vary widely in residential/commercial buildings.

Figure 18 shows common vertical configurations of infill buildings. Many have infill walls that either continue to the ground story or have partially or mostly open ground stories to accommodate shops or parking. In some areas, infill buildings may have basements or may be constructed on sloping sites with longer columns on one side of the building.

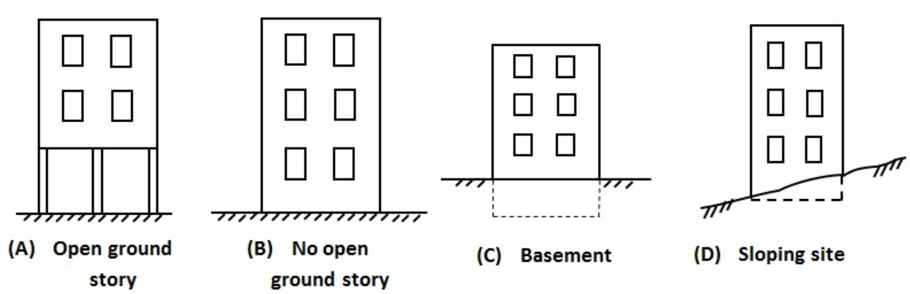


Figure 18. Common vertical configurations of infill buildings

Types of Infill Walls

A number of different types of infill materials are used throughout the world. The type of infill used in a particular location depends on the type and quality of materials available in that location, as well as the skills of local workers. Table 2 shows different types of infill wall material present in the World Housing Encyclopedia reports.

Table 2. Types of masonry infill wall material

Solid fired clay brick	
Perforated fired clay brick	
Hollow fired clay brick (hollow clay tile)	
Semi-dressed stone	 

Solid concrete block	
Lightweight autoclaved aerated concrete (AAC) block	
Hollow concrete block (ungrouted or grouted)	

Photo credits: Svetlana Brzev, British Columbia Institute of Technology (solid, perforated and hollow fired clay brick, dressed stone masonry); L. Thomas Tobin, GeoHazards International (solid concrete block); Building Concrete Masonry Homes: Design and Construction Issues, U.S Department of Housing and Urban Development Office of Policy Development and Research (hollow concrete block).

Infill masonry walls are commonly built with cement-based mortar. Lime is also used in many countries. Mortar quality can vary greatly from place to place. Examples in the World Housing Encyclopedia show weak mortars with cement-to-lime-to-sand ratios ranging from 1:0:6 to 1:0:8 (in volume) are used for some buildings in South Asia, though stronger mortars are used in other buildings in the region (i.e., Sarangapani, 2002).

Infill walls are typically solid but can be built using cavity construction (Figure 19), where inner and outer wythes of masonry enclose a cavity with a width typically between 50 mm and 150 mm. This technique is used with all types of masonry. The airspace between the wythes helps to insulate the building and acts as a barrier to water when detailed properly. In many cavity walls, also called noncomposite walls, thermal insulation is placed between the wythes to further enhance thermal efficiency. The cavity improves thermal performance but also makes the wall weaker under earthquake loading than a double-wythe wall with adequate bond courses. Each wythe of a cavity wall must resist loading independently and has a larger height-to-thickness ratio than a double-wythe wall, making each wythe more prone to out-of-plane failure. Wall ties prescribed in some codes (for example, NZS 4230:2004 and TEK 16-1A, 2005) can improve the seismic performance of cavity walls.

Figure 19. A cavity wall (photo on the left from Syracuse University libraries, photo on the right from anewhouse.com.au)

1.4 Beneficial Characteristics of Infill Walls

People construct buildings using masonry infill walls for a number of reasons. Masonry infill walls provide thermal insulation, particularly in hot climates, and a moisture barrier. They provide an acoustic barrier between spaces and from outside noise, which is especially important in residential construction. Masonry walls provide an effective fire barrier, and when made of brick or concrete block, they provide security and protection from intruders, especially in residences.

Masonry units are often readily available, inexpensive, and supplied by local manufacturers using local labor and often local materials. Transportation distances from the manufacturer to the local shops selling building materials or to the construction site itself tend to be short, further reducing costs. Other wall materials, if available, are often more expensive. Although infill construction is labor intensive, labor tends to be inexpensive in countries where these buildings are popular. Portland cement, which can be used for mortar and the masonry itself, may be relatively expensive in some countries. Masonry materials can be environmentally friendly compared to some alternatives, especially if the masonry is made locally. Reduced transportation distances lessen the environmental effects of pollution created by diesel engines that power the trucks, trains and ships used to transport construction materials.

Lastly, infill walls—if detailed properly—contribute to a structure’s lateral-force-resisting capacity and increase its energy dissipation capacity. In addition, infill walls increase a building’s initial lateral stiffness and decrease its initial vibration period, which can result in better performance in low-to-moderate earthquake shaking.

2 Selecting a Strategy to Resist Earthquake Effects

2.1 Overview of Design Strategies

This document presents five strategies that designers can use to improve the seismic performance, economy and reliability of reinforced concrete frames with infill. These design strategies are:

A. Design infill panels as structural members that act as diagonal compression struts to resist lateral demands
B. Provide a rocking spine that eliminates the commonly encountered weak-story mechanism
C. Add elements to the ground story to create a strong, ductile, or energy dissipating base that prevents weak-story collapse
D. Separate the infill panels from the frame while preventing out-of-plane failure, and then design the frame as a bare frame
E. Change the structural system to shear walls or confined masonry

Each strategy is described in a separate section of the document. Each section describes the design strategy, gives rules of thumb for determining when the strategy will and will not be effective, and explains essential design concepts. This document does not cover the design strategy used in Eurocode 8—which is to mitigate the adverse effects of infill with special additional detailing in the concrete frame—because Eurocode 8 already codifies this strategy. In addition, in an attempt to only introduce economical strategies, this document does not cover currently available advanced and typically expensive protective systems such as base isolation, active control, hybrid combinations of active and passive energy devices, or tuned mass and liquid dampers.

2.2 Considerations for Selecting a Design Strategy

When determining which design strategy to use, the design engineer should consider the factors described in the sections that follow.

Economics, Local Construction Practices and Regulatory Environment

Consider if a strategy is likely to be too expensive, such as because of constructability constraints or a lack of readily-available materials. Assess the local construction practices and determine whether builders can actually construct the design strategy for a reasonable price. Surveying local builders and determining material prices can help determine which strategy is more likely to be most economical. The local regulatory environment may also play a role in determining the feasibility of certain design strategies. Local ordinances or regulations that prescribe ground floor conditions (such as pedestrian walkways or breezeways) may limit design options. The flexibility of the local building authorities and their willingness to review and approve designs that are new to them should be taken into account. This consideration is particularly important for approaches such as a rocking spine, which rely on design concepts that are unfamiliar to most building authorities.

Building Use, Size, Configuration, Distribution and Type of Infill Walls

The building's physical characteristics have perhaps the greatest impact on selecting the most appropriate design strategy for an infill building. Tall buildings require different strategies than short buildings. Tall, slender buildings will have different dynamic characteristics and will require a different strategy than tall, wide buildings. Buildings with open ground stories or eccentrically distributed infill walls require solutions that will prevent unacceptable damage from concentrating in one place. A building with many heavily perforated infill walls will behave differently than a similarly shaped building with many solid walls, so the design strategy may need to be different. A school with many partial-height infill walls for classroom windows will require a strategy that specifically prevents the infill walls from creating captive columns.

Infill Material(s) and Properties

The type of infill used in local construction practice, and its properties, will have a significant impact on the appropriateness of each design strategy. For example, weak infill materials, such as hollow clay tile, are unlikely to have enough strength to be used as reliable diagonal compression struts, unless the level of expected ground shaking is very low.

Seismic Hazard

The expected size of the design earthquake and return period will influence the selection of a design strategy. In areas where the expected shaking is moderate, the design forces will be moderate and it may be easy to achieve an acceptable level of performance by including ordinary infill walls as diagonal compression struts. In areas where strong shaking is expected, additional reinforced concrete or steel elements may be necessary to provide sufficient lateral strength. In hazard zones where the expected return period is long, or the size of the design earthquake is moderate or small, more elaborate design strategies may not be warranted.

Desired Seismic Performance

Many building codes intend to preserve the lives of building occupants in the design earthquake but permit substantial building damage in order to reduce the cost of construction. Even a standard code-designed building may be a complete economic loss and may need to be demolished following a design-level earthquake. If everyone in the building at the time of the earthquake was able to exit safely, then the building met the intent of the code. A number of codes also intend to prevent structural collapse but accept some loss of life in an earthquake that is larger and less likely than the design earthquake. These realities of code-level building performance can come as a shock to building owners.

Depending on the type of facility, the owner may want the building to perform better than a standard building in the expected design earthquake. For example, it may be necessary to limit damage in addition to preserving the lives of occupants. If the building is a hospital, fire station, or other facility necessary for immediate post-earthquake response, it will need to be functional following the design earthquake. This means that damage to the structural system and architectural shell (ceilings, partitions, etc.) should not prevent use of the building; and utilities, such as water and electrical power, should be available post-earthquake. A higher level of seismic performance will require a design strategy that provides greater strength and global stiffness in order to limit damage to an acceptable level. Achieving a higher seismic performance may require the use of protective systems, such as supplemental damping or base isolation (not covered in this document).

Most codes regard building performance as a combination of the performance of the structure and all the other parts of the building, such as the architectural finishes, plumbing and mechanical systems, collectively referred to by many as nonstructural components. ASCE 41, a U.S. performance-based design standard, and its freely available pre-standard, FEMA 356, define building performance levels based on the amount of damage caused to the structural and non-structural components. Figure 20 schematically illustrates the three main performance levels: Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). Immediate Occupancy means that very limited structural damage has occurred and the building can be occupied immediately. Life Safety means that significant structural damage has occurred, but the risk of life loss is low. Collapse Prevention simply means that the building has not collapsed but may be on the verge of partial or total collapse.

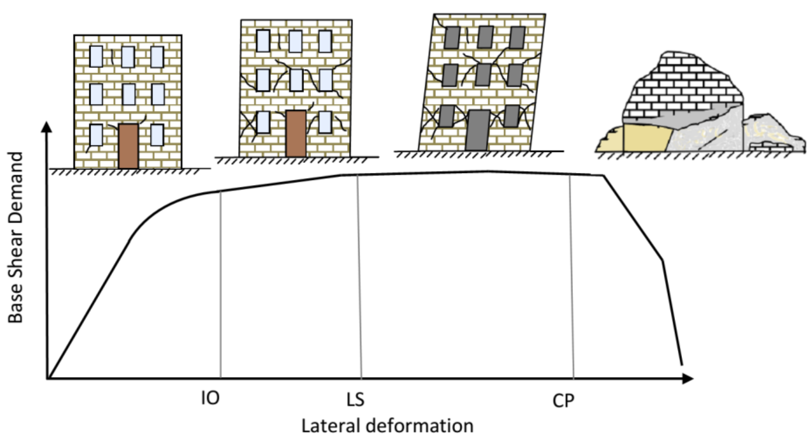


Figure 20. Schematic illustration of different Structural Performance Levels. IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention (after Ron Hamburger)

Design Strategies A-D in this guide are primarily intended to provide Life Safety or Collapse Prevention performance. Buildings that require higher performance may necessitate the use of seismic protective systems (outside the scope of this guide), though in some cases reinforced concrete shear walls (Design Strategy E) may be able to provide the desired seismic performance.

General Guidance on Selecting a Design Strategy

Each building is unique, and the designer should select the most appropriate strategy on a case-by-case basis. Despite this, when considering the factors discussed in the sections above it becomes apparent that certain strategies will not be as effective – or may not be appropriate – for buildings with certain configurations or characteristics. Table 3 provides general guidance on when each strategy is more appropriate, less appropriate, or not appropriate at all (in some cases).

Table 3. Selecting a Design Strategy

Design Strategy	More appropriate for:	Less appropriate for:	Not appropriate for:
A: Infill as key part of lateral system	Low to moderate seismic hazard Shorter, wider buildings Regular buildings Buildings with substantial infill in ground story	High seismic hazard Taller, more slender buildings	Buildings with few infill panels in ground story compared with upper stories Buildings with severe vertical or torsion irregularities
B: Rocking spine	Moderate to high seismic hazard Taller or more slender buildings Buildings with few infill panels in ground story compared with upper stories	Small, simple, regular buildings	
C: Strong/ductile base	Moderate to high seismic hazard Buildings with few infill panels in the ground story compared with upper stories	Buildings with few infill panels in the upper stories compared with ground story	Upper stories have a torsional irregularity
D: Separate panels from frame	Moderate to high seismic hazard Taller or more slender buildings Buildings with few infill panels in ground story compared with upper stories Buildings with other irregular configuration of infill panels	Smaller, simple, regular buildings	
E: Change structural system to: Confined Masonry	Smaller low-rise buildings Regular buildings Buildings with substantial infill in ground story Concrete construction quality in the region is poor	Buildings with few infill panels in ground story compared with upper stories Very poor masonry material quality Infill panels have many openings	Tall buildings (more than 6 stories)
Reinforced Concrete Shear Wall	High seismic hazard or higher seismic performance desired Taller buildings Buildings with few infill panels in ground story or infill panels with many openings	Smaller, simple, regular buildings Floors and/or roof are of light framed construction	

2.3 Construction Considerations

As with any type of building, concrete frames with masonry infill must be built properly, using good construction practices. The builders must use quality materials, not only for the infill masonry and mortar, but also for the concrete and reinforcing steel in the building frame. The workmanship should be of high quality. For accountability reasons, construction should be inspected to ensure the engineer's design was followed. Ample guidance on good construction practices for concrete buildings are found in national codes and specifications, as well as in documents such as the World Housing Encyclopedia tutorial on reinforced concrete frames with infill walls, *AT RISK: The Seismic Performance of Reinforced Concrete Frame Buildings with Masonry Infill Walls* (Murty et al, 2006), available from the World Housing Encyclopedia website (www.world-housing.net).

3 Design Strategy A: Infill Panels as Structural Members

3.1 Motivation and Design Rationale

Building behavior in past earthquakes suggests that infill panels can function as de facto shear walls and help to prevent collapse of non-ductile concrete frames (*e.g.,* Goel, 2001). Figures 21-23 show some examples from the 2001 Gujarat, India earthquake. Infill walls constructed of relatively strong infill materials, such as good quality clay brick or solid concrete brick, can resist significant loads before failing. This design strategy considers the infill panels as structural members, captures their effect on the building structural response, and deliberately designs the panels to prevent formation of a weak story during shaking.

Figure 21. Diagonal shear cracks in the wall indicate that it provided resistance and energy dissipation during the 2001 Gujarat, India earthquake. (Photo and description by Rakesh K. Goel, California Polytechnic State University)

Figure 22. Infill panels provided stability to the overall structure during the 2001 Gujarat, India earthquake. (Source: Patel et al., 2001)

Figure 23. Infill panels prevented collapse of the whole structure during the 2001 Gujarat, India earthquake. (Source: Patel et al. ,2001)

The infill walls in the examples shown in Figures 21-23 were unlikely to have been explicitly designed to resist lateral loads. The arrangement and strength of these infill walls compared to the level of shaking led to a successful outcome. However, as noted in prior sections, had the strength been lower (leading to weak stories), had the infill arrangement concentrated deformations in a few columns, or had the infill wall forces resulted in shear failure of columns, the outcome could have been much worse. An explicit, rational design process is necessary in order to utilize infill panels as part of the lateral system, or at minimum to ensure that they do not create problems. Fortunately, it is not difficult to harness the strength and stiffness of infill walls through a rational design process.

3.2 Concepts and Implementation Strategies

Basic statics shows that in an infill building, the ground floor infill walls will have the highest forces because they must resist the lateral forces from all of the floors above (Figure 24). If all infill walls have the same size, thickness and material properties, the ground floor infill walls typically—but not necessarily—fail first during seismic loading. (Upper stories could fail before the lower stories if, for example, higher modes are heavily activated during ground shaking, causing the upper stories to exceed their deformation capacities before the lower stories.)

A building with the same infill walls all the way down to the ground level may still form a weak story during shaking, due to the failure of the infill. Weak stories may even occur at intermediate stories if infill walls in these stories fail, such as due to in-plane and out-of-plane interaction effects. Recent publications show this phenomenon both during recent earthquakes (Urich and Beauperthuy, 2012; Mosalam and Gunay, 2013) and in analytical simulations (Gunay et al., 2009). Accordingly, infill walls in each story should be designed to be strong enough to resist the expected level of shaking without brittle failure. Capacity design[2] should be used to eliminate undesirable failure modes, such as column or joint shear failure caused by frame-infill interaction, by ensuring that the columns and joints are strong

[2] Capacity design is an approach to designing structural systems that permits inelastic behavior, or action, in inherently non-critical ductile elements (such as beams) rather than columns, and in certain failure modes within elements (such as ductile flexural failure) rather than brittle shear failure. For example, a beam's nominal shear strength should be designed to be greater than the shear corresponding to a beam's plastic moments at each end.

enough to resist the forces created by infill strut action. Also, the designer must control the relative strengths of the building's stories to reduce the chances of a single-story collapse mechanism; subsequent sections of this manual provide guidance.

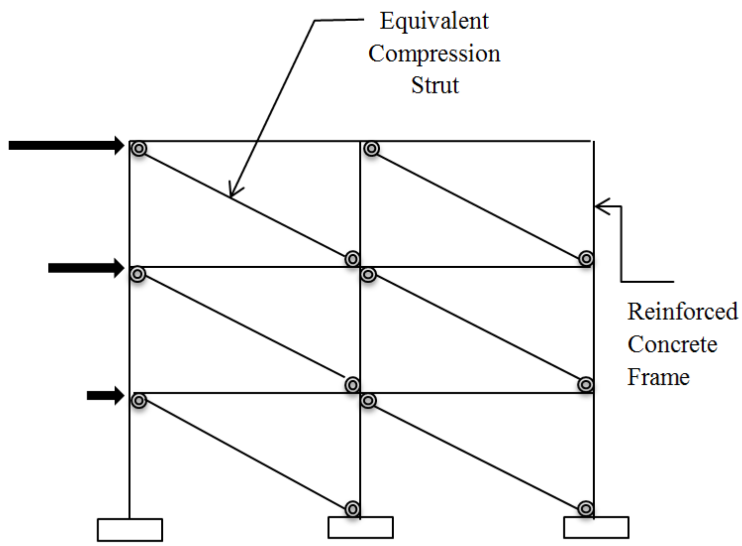


Figure 24. Building with infill panels modeled as equivalent compression struts.

Determining the Required Strength and Minimum Properties for Infill Walls

The structural response of RC buildings with masonry infill significantly depends on the mechanical properties of the masonry. While very weak masonry infill fails relatively fast under seismic load, strong masonry infill can significantly increase the lateral strength of the frame, as reported by many researchers (e.g., Uva and Fiore, 2012). Masonry material properties (such as compressive strength, shear strength, and modulus of elasticity) are highly variable from region to region. Some standards, such as Mexico standard NTC-M (2002), Turkish Earthquake Code (2007), and ASCE 41/FEMA 356 suggest lower bound values or recommended values for masonry properties. While these recommended values may be practical for some regions, they may be too conservative or unconservative depending on the quality of the materials used. Therefore, it is highly recommended that for design purposes, the typical properties of local masonry given in local codes and/or databases be used. A list of some useful references for properties of masonry material is provided in Appendix A.

Openings in Infill Panels

It is important that a designer considers the effects of any openings in infill walls. Sizeable openings in infill panels will reduce the lateral stiffness and lateral strength of the infilled frame. Asteris (2003) showed that panels with 50% or more of the panel area open do not contribute much lateral stiffness, and such frames essentially behave the same as a bare frame. Reductions in strength and stiffness –

which may be different – vary with opening size and position within the panel. Openings adjacent to columns have the potential to create captive columns. (See the discussion on captive columns earlier in this document.) In particular, partial height infill walls across the entire bay width, such as those commonly seen below windows in schools, should not be used unless the infill is isolated from the frame as discussed in Strategy E in this document. Designers planning to rely on the formation of equivalent compression struts in a building's lateral system, as in this Strategy A, should ensure that any openings are outside of the strut. Furthermore, panels with 50% or more of the panel area open should be modeled as open, rather than with struts.

Guidance on Modeling Infill Walls as Equivalent Diagonal Struts for Structural Analysis

Engineers often model solid infill panels as equivalent compression struts for structural analysis, in order to simplify the model and reduce computational effort. Different researchers have proposed numerous models (e.g., Stafford Smith, 1962; Paulay and Priestley, 1992; Sanainejad and Hobbs, 1995). Single strut models (Figure 24) are the simplest to use and are sufficiently accurate to capture the global behavior problems that often result when infill is ignored. This section summarizes procedures for determining the strut characteristics for clay brick masonry and concrete block infill from recent US standards, *Building code requirements and specifications for masonry structures (TMS 402-11/ACI 530-11/ASCE 5-11)* and *Seismic Evaluation and Retrofit of Existing buildings (ASCE 41-13,* and its freely available precursor document, *FEMA 356)*. Similar relations can be developed for other types of infill. The latter document recommends that designers use a single strut for panels with a length to height ratio of 1.5 or less, and two parallel struts sloped at 45 degrees for panels with larger aspect ratios. In the two-strut case, the equivalent force is distributed between them.

According to section B.3.4.1 of TMS 402-11, the thickness of the equivalent diagonal compression strut is the net thickness of the infill, its elastic modulus is the elastic modulus of the infill, and its equivalent width can be calculated as follows[3]. Equations are numbered with the letter of the design strategy to which they belong following the equation number; Equation 1 under Strategy A is numbered (1a).

$$w_{inf} = \frac{0.3}{\lambda_{strut} \cos \theta_{strut}} \qquad (1a)$$

$$\lambda_{strut} = \left(\frac{E_m \, t_{net,inf} \sin(2\theta_{strut})}{4E_{bc} \, I_{bc} \, h_{inf}} \right)^{0.25} \qquad (2a)$$

Where

[3] The equations presented here for calculating the strength and properties of the equivalent compression strut have their origin in research dating back to the 1960s. According to commentary in section B.3.4.3 of TMS 402-11, the method proposed in this document is attractive because of its relative simplicity and accuracy. For example, Flanagan and Bennett (2001) examined accuracy of this method to predict the strength of clay tile, clay brick and concrete masonry infill in steel and concrete bounding frames. They found the coefficient of variation of the ratio of measured to predicted strength of the infill to be 21%.

w_{inf} thickness of the equivalent strut (mm)

E_m modulus of elasticity of masonry in compression (MPa)

$t_{net,inf}$ net thickness of the infill (minimum total thickness of the net cross-sectional area, mm)

E_{bc} modulus of elasticity of bounding columns (MPa)

I_{bc} moment of inertia of bounding column for bending in the plane of infill (mm^4)

h_{inf} vertical dimension of infill (mm)

θ_{strut} angle of infill diagonal (degrees) with respect to the horizontal (Figure 25).

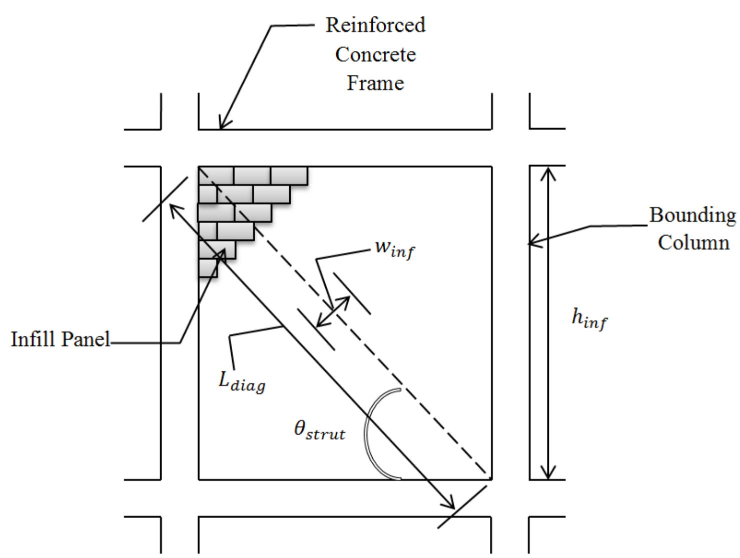


Figure 25: Key parameters for modeling infill as an equivalent compression strut

The axial stiffness of the equivalent strut can be derived using the following equation,

$$k_{inf} = \frac{E_m w_{inf} t_{net,inf}}{L_{diag}} \qquad (3a)$$

in which L_{diag} (mm) is the diagonal length of the infill panel, which is also the length of the diagonal strut. For panels with openings, ASCE 41 suggests the following equation for calculating the stiffness of infill panels:

$$K = \left(1 - 2\frac{A_{op}}{A_{tot}}\right) K_{solid} \qquad (4a)$$

where A_{op} is the opening area, A_{tot} is the total infill area, and K_{solid} is the stiffness of infill without the opening(s). The authors of this guideline recommend that when using this equation the panel should be less than 50% open, and any openings should lie outside the strut. According to ASCE 41-13, the expected in-plane, infill shear strength in Newtons (N), V_{ine} is calculated using the following equation:

$$V_{ine} = A_{ni} f_{vie} \qquad (5a)$$

In which A_{ni} is area of the net mortared/grouted section across the infill panel (mm^2), and f_{vie} is the expected shear strength of the bed joints (MPa). The expected shear strength of existing infills, f_{vie}, shall not exceed the expected masonry bed-joint shear strength v_{me}.

Alternately, the expected in-plane, infill shear strength in N, $V_{n,\inf}$,can be calculated in accordance with section B 3.4.3 of TMS 402,where $V_{n,\inf}$ and V_{ine} (referred to earlier) both represent the in-plane shear strength of masonry infill. The different notations are used here to keep the formulas consistent with the referenced documents. According to section B 3.4.3 of TMS 402, $V_{n,\inf}$ is calculated as the smallest of (a), (b) and (c) as follows:

a) $(150mm)t_{net,inf} f'_m$,where $t_{net,inf}$ is in mm and f'_m refers to the compressive strength of the masonry and is in MPa (6a)

b) The calculated horizontal component of the force in the equivalent strut at a horizontal racking displacement of 25 mm (1.0 in)

c) $V_n/1.5$ where V_n is in N (7a)

where V_n is the smallest nominal shear strength from Section 3.2.4 of TMS 402-11, calculated along a bed joint of the equivalent frame and taken as the smallest of:

a) $0.33 A_n \sqrt{f'_m}\ in\ N$, where f'_m is in MPa (8a)

b) $0.83 A_n\ in\ N$ (9a)

where A_n is the net cross-sectional area of the infill (in mm^2).

The strength of the infill panel may be controlled by several different failure mechanisms described by the equations above. The strength may be controlled by the panel's resistance to compression failure, often referred to as "corner crushing" or by shear failure along a bed joint. For panels with length to height ratios greater than 1.5 being modelled with two parallel 45 degree struts, the strength may also be controlled by the bearing capacity of the infill as it compresses against the bounding frame. The two parallel struts initiate at the top of the column on the panel's left side and the bottom of the column on the panel's right side, if the equivalent seismic forces are applied from left to right. The bearing height of the strut (i.e., the distance over which the infill panel compresses against the bounding frame) on the columns and beams can be reasonably assumed as one third of the infill panel's height (ASCE 41-13). Consequently, the bearing (compressive) strength of the infill is obtained by:

$$F_{mc} = f'_m \left(\frac{h_{inf}}{3}\right) t_{net,inf} \qquad (10a)$$

This bearing strength must be compared against the panel's in-plane, infill shear strength previously calculated (see equations above) in order to determine the strength of the panel. The designer should compare the expected compressive strength of the infill to the axial force in each strut. The axial force will be more reliable if determined using nonlinear static analysis, though it is possible to determine the axial force demands through careful linear analysis. If the expected compressive strength of the infill is smaller than the axial force in each strut, the infill will need to be made stronger, or more panels added, to provide sufficient strength. However, it is also necessary to make sure that the infill is not so strong that it will cause shear failure in the columns or beam-column joints. The designer should check columns and beam column joints to ensure that they can resist the strut forces without failing in shear.

The strut model explained above assumes that the infill panel does not rely upon mechanical connectors to transfer in-plane load (TMS 402-11). Without mechanical connectors to transfer load, separation of the infill and frame occurs under small lateral deformations and initiates nonlinear behavior in the structure. Horizontal steel anchors that tie the wall to the adjacent columns, as shown in Figure 26, can ensure force transfer between the infill and frame (Murty et al., 2006). However, this will change the behavior of the system, and the mentioned strut model may no longer be applicable. Anchoring the infill to the surrounding frame reduces dislodging problems and provides more out-of-plane anchorage and a more uniform hysteresis loop. However, anchoring the infill will change the amount and profile of the force transferred from the infill to a column, thus changing the requirements for the column's dimensions and reinforcement, especially if the infill is strong relative to the frame.

Differences of professional opinion exist as to whether infill panels should be mechanically connected to the frame. Eurocode 8 and a number of researchers encourage the use of connectors to help prevent out-of-plane failure. However, others contend that connectors are difficult to construct properly so that they do not can cause premature damage along the boundaries of the infill under in-plane loading. Such damage reduces the out-of-plane strength of the infill by reducing the infill's ability to achieve arching action. Using this rationale, TMS 402-11 discourages using mechanical connections between an infill and frame, stating that "while mechanical connections, including the use of reinforcement, are permitted, they must be detailed to preclude load transfer between the infill and bounding frame." Thus, anchoring the infill to the frame must be done carefully. Also, infill walls are more difficult to build with mechanical connectors to the frame, so infill walls in many countries are most often built without them.

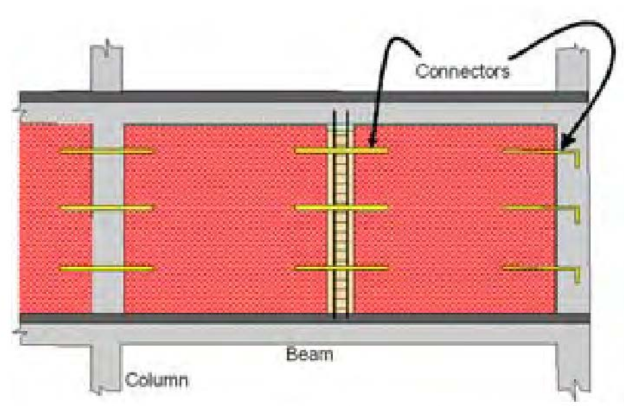


Figure 26. Details for anchoring the infill panels to the frame (Murty et al., 2006).

Practical Limits on Height and Configuration to Reduce Chances of Weak Story Mechanism, Given Infill Wall Strength and Seismic Demand

As previously described, the strength of the infill walls and frame at any story limits the maximum story shear in that story. In a multi-story frame completely filled with identical infill panels from top to bottom, the ground-story panels typically will have the highest in-plane demand compared to their strength capacity. To avoid a weak story mechanism caused by failure of the ground-story panels, the base shear should be less than the total strength of the ground-story infill panels multiplied by demand-to-capacity factor of three. Based on experimental testing, ASCE 41-13 suggests that infill panels have some capacity in the inelastic range, taking some time to degrade, and permits component demand modification factors ("m-factors") of 3 for low-strength frames for life safety.

To calculate demand-to-capacity base shear for identical infill panels:

$$3n\Phi V_{inf} > V_b \qquad (11a)$$

where n = number of panels and V_b = unreduced design base shear (W S_a / g) for the largest earthquake for which the designer is trying to achieve a reasonable margin against collapse. Some codes (i.e., BIS 2002) refer to this large earthquake as the MCE (maximum credible earthquake). Using the earthquake intensity for which collapse prevention is desired is a more general procedure. Other procedures that design for life safety in the "design basis level of earthquake" (DBE) use associated design factors that can vary significantly from country to country. Because making the infill panels too strong may cause shear failure in the columns or joints, the expected infill panel strength should be used, with the reduction factor Φ =1.0.

For new buildings where one story will be open or relatively open (generally the ground story), avoiding a weak story mechanism requires additional design attention and may necessitate a change of design strategy, such as to strategies C or B in this document. Strategy C introduces a smaller number of strong elements in the open story to prevent a weak story mechanism from forming. Strategy B introduces a rocking spine to prevent the formation of a weak story. Some codes, such as ASCE 7-10[4], prohibit weak or soft stories in areas of high seismic hazard and require the designer to calculate story-to-story variations in strength and stiffness. Other codes, however, contain only simple provisions for strengthening the concrete frame members in the open story that are intended to prevent the formation of a weak story. For example, the Indian seismic code (BIS, 2002) requires frame members in

[4] In order to avoid a weak story mechanism, vertical irregularities need to be prevented. According to ASCE 7-10, weak story irregularity exists where the story lateral strength (total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration) is less than 80% of that in the story above. This is not allowed for high seismic areas (categories E and F as defined ASCE 7-10). Also, an extreme weak story irregularity exists when this value reduces to 65%. In this case, the structural height of the building shall not be more than two stories or 30 ft (9 m). Some other codes define soft and weak stories slightly differently.

the open first story to be designed for 2.5 times the design seismic forces, and Eurocode 8 recommends increasing the design forces for the soft ground-story columns 1.5 to 4.68 times, depending upon several factors. The Bulgarian Seismic Code (1987) requires that seismic design forces for soft stories in masonry infill reinforced concrete frames be increased by two times the corresponding design forces for a regularly infilled frame and by three times the design seismic forces for a regular bare frame (Kaushik et al., 2006).

Provisions for strengthening concrete frame members in open stories should be used with caution, because strengthening the frame members in only the weak story may reduce, but not eliminate, the stiffness irregularities that cause deformations to concentrate (Murty et al., 2013). Recent research (Padhy et.al, 2012) has shown that the strengthened frame members may not provide adequate protection against collapse. Even if using code provisions for strengthening open-story frame members, designers must model the infill walls and should verify via computer analysis that a weak story will not form. In addition to verification via analysis, the designer can select the story strength provided by infill panels in order to prevent weak story mechanism.

The following “rule of thumb” ratio can be used to indicate if a weak story mechanism occurs at story *i*:

$$\frac{strength\ of\ story\ i}{strength\ of\ \ story\ i{+}1} > \frac{n+1-i}{n-i} \qquad \text{(12a)}$$

in which *n* is the number of stories in the building, counting the ground story as 1, second story as 2, and so on (*n*=5 is a ground-plus-four-story building). Note, for example, that the first floor of a four-story building has a balanced strength ratio of 4/3 = 1.33 and the second story of the same building has a ratio of 3/2 = 1.5.

Arrangement of Infill Walls in Plan to Limit Torsion; Redundancy and Irregularity

Irregular placement of infill walls in plan creates torsion and leads to increased shear demand in the walls and frame members. The main way to reduce this effect is to limit the eccentricity between the center of mass (i.e., the point representing the mean location of the distribution of mass in the building or on a floor of a building) and the center of stiffness (i.e., the point representing the mean location of the distribution of stiffness in the building or on a floor of a building) in each direction, by appropriate arrangement of infill walls in plan.

Several codes provide guidance on restricting the amount of eccentricity to reduce plan irregularities in frames with masonry infill (Kaushik et al., 2006). Nepal’s code (NBC-201, 1995) limits the eccentricity between the center of mass and center of stiffness to 10% of the building dimension along each direction. The Costa Rican code (1986) limits this value to 5% and 30% of the building dimension in each direction for regular and irregular structures, respectively (Kaushik et al., 2006).

Eurocode 8 (2003) does not require infill panels to be modeled if the arrangement of infills in plan is not severely irregular, but instead permits a doubling of the accidental eccentricity. For severely irregular arrangement of infills, such as corner buildings with solid property line walls on the non-street sides (Figure 17), Eurocode 8 requires a three-dimensional dynamic analysis with the infills modeled, along with a sensitivity analysis to examine the effects of infill position and stiffness. This provision creates an incentive for designers to avoid severely irregular buildings.

Limits on Infill Panel Aspect Ratio and Construction Details to Prevent Out-of-Plane Failure

During earthquakes, infill panels can be subjected to large forces perpendicular to their plane. These out-of-plane forces can be especially large in the upper stories because the building's response amplifies accelerations. Recent earthquakes provide many examples of infill panels that suffered out-of-plane failure and were expelled from the frame. Infill walls in full contact with the frame on all sides and meeting certain other criteria will resist out-of-plane loading through arching action and will not need any out-of-plane anchorage to frame members. Guidance on arching action given in FEMA 356 notes that arching action shall be considered only if all the following conditions apply:

1. The panel is in full contact with the surrounding frame components.

2. The product of the elastic modulus, E_f, times the moment of inertia, I_f, of the most flexible frame component surrounding the panel exceeds a value of 3.6×10^9 lb-in^2 (10MN-m^2).

3. The frame components have sufficient strength to resist thrusts from arching of an infill panel.

4. The h_{inf}/t_{inf} ratio, where h_{inf} is infill panel height and t_{inf} is the infill panel thickness, is less than or equal to 25.

Most contact problems occur at the top of the panel because the beam above the panel is generally built before the infill is placed. Therefore, it can be difficult to fully fill the gap between the top course of masonry and the beam, and even if the space is filled during construction, shrinkage of the mortar over time can open a small gap. Full contact at the top can be achieved by doing all of the following:

- the top mortar joint between the wall and frame must be tightly filled with mortar;
- mortar should be of good quality and the same as used in the rest of the wall; and
- the top mortar joint should have a reasonable thickness.

In cases where the infill panel material is weak (*i.e.,* hollow clay tile), full contact may not be desirable because beam deformation due to creep and additional loads imposed later can crack the infill. A thin deformable layer such as expanded polystyrene (EPS) or cork (used in Portugal) can prevent cracking of the infill, but arching action may not develop if the deformable layer is too thick.

Panels not in full contact will resist out-of-plane loading through bending, and unreinforced panels have very little bending strength. Recent research (Dawe and Seah, 1989) indicates that panels that are not in full contact at the top may have a small additional amount of capacity from horizontal arching action, provided the gap is sufficiently small. In practice, this means that most unreinforced panels not in full contact will need to be anchored to adjacent frame members to prevent out-of-plane failure. It is much less costly to follow the procedures outlined above during construction to achieve full contact.

In general, the acceptable slenderness ratio (ratio of the smaller of length or height to thickness) depends on many factors including properties of the infill material, quality of mortar, seismicity of the building site, and position of the infill wall in the building. Several standards provide guidance on acceptable slenderness ratios. Eurocode 8 notes that particular attention should be paid to preventing out-of-plane collapse of masonry panels with a slenderness ratio greater than 15. ASCE 41-06/ FEMA 356 contains guidance on infill panel height-to-thickness (h_{inf}/t_{inf}) ratios (Table 4) for different levels of seismic hazard and design performance[5]. Unreinforced infill panels with h_{inf}/t_{inf} ratios less than the specified values and meeting the requirements for arching action need not be analyzed for out-of-plane seismic forces.

Table 4.Maximum h_{inf}/t_{inf} ratios for which out of plane analysis is not required (FEMA 356))[5]

Table 7-10 Maximum h_{inf}/t_{inf} Ratios[1]

	Low Seismic Zone	Moderate Seismic Zone	High Seismic Zone
IO	14	13	8
LS	15	14	9
CP	16	15	10

1. Out-of-plane analysis shall not be required for infills with h_{inf}/t_{inf} ratios less than the values listed herein.

Walls that meet the criteria for arching action but do not meet the ASCE 41 h_{inf}/t_{inf} ratios may still have adequate resistance to out-of-plane failure, which must be determined by calculating the wall capacity using standard relationships for arching action, such as those below (from Flanagan and Bennett 1999, now codified in TMS 402):

$$q_{n\,\mathrm{inf}} = 105\left(f'_m\right)^{0.75}\, t_{\mathrm{inf}}^2 \left(\frac{\alpha_{arch}}{l_{\mathrm{inf}}^{\;2.5}} + \frac{\beta_{arch}}{h_{\mathrm{inf}}^{\;2.5}}\right)$$

where $\alpha_{arch} = \dfrac{1}{h_{\mathrm{inf}}}\left(E_{bc}\, I_{bc}\, h_{\mathrm{inf}}^2\right)^{0.25} < 35$ and $\beta_{arch} = \dfrac{1}{l_{\mathrm{inf}}}\left(E_{bb}\, I_{bb}\, l_{\mathrm{inf}}^2\right)^{0.25} < 35$ (13a)

I_{bb} moment of inertia of the bounding beam for bending in the plane of the infill

I_{bc} moment of inertia of the column for bending in the plane of the infill

[5] See the Desired Seismic Performance section for a description of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) performance objectives. FEMA 356 defines zones of high seismicity as those where the 10%/50 year, short-period response acceleration, S_{xs}, and the 10%/50 year, one-second period response, S_{x1}, are greater than or equal to 0.5g and 0.2 g, respectively. Zones of moderate seismicity are defined as: $0.167g \le S_{xs} < 0.5g$ and $0.067g \le S_{x1} < 0.2g$. Zones of low seismicity are defined as: $S_{xs} < 0.167g$ and $S_{x1} < 0.067g$.

E_{bb} elastic modulus for the bounding beam

E_{bc} elastic modulus for the column

l_{inf} infill panel length

h_{inf} infill panel height

t_{inf} infill panel thickness

When a side gap is present, α_{arch} is set to zero, and when a top gap is present β_{arch} is set to zero.

Because a double-wythe brick wall is usually 210-250mm, the maximum infill panel height for which the design engineer could avoid checking the panel for out-of-plane seismic forces using ASCE 41 is 2.16 to 2.25 m. The current literature does not contain a consensus on an appropriate upper limit on h_{inf}/t_{inf} ratios. For example, a recent literature review by Tremayne et al. (2012) found that ASCE 41's h_{inf}/t_{inf} limits can be unconservative in some cases. In contrast, Fardis (2000) reported that for infill walls with slenderness ratios as high as 30 but without openings, the combination of in-plane and out-of-plane demands under bi-directional ground motions is not a major problem, but further studies are required for infill walls with openings. TMS 402-11 also permits h_{inf}/t_{inf} ratios of up to 30 for participating infills (those not isolated from the frame).

In-Plane (IP) – Out-of-Plane (OOP) Interaction

Because earthquake ground motions are multi-directional, infill walls will experience simultaneous in-plane and out-of-plane demands. An interaction effect exists between the in-plane strength of the wall and the out-of-plane strength, where forces in one direction reduce the strength in the other direction to an extent, depending on the aspect ratio of the infill panel. Reductions in out-of-plane strength may be as high as 50% for panels with high aspect ratios (Angel et al., 1994), while in-plane strength can be reduced by a similar amount (Flanagan and Bennett, 1999). In spite of these observations, this interaction is generally ignored in current engineering practice.

Because slender infill panels with large h/t ratios are more susceptible to out-of-plane damage that reduces in-plane strength, infill panels that do not meet the h/t ratios given in Table 4 should be checked for in-plane and out-of-plane interaction. Kadysiewski and Mosalam (2009) proposed a model for considering both the in-plane and out-of-plane response of solid infill panels, as well as the interaction between IP and OOP capacities.

The simple interaction equation, which can be used to compute the reduced in-plane capacity due to out-of-plane forces (and vice versa), is as follows:

$$\left(\frac{P_{ip}}{P_{ipo}}\right)^{3/2} + \left(\frac{M_{oop}}{M_{oopo}}\right)^{3/2} \leq 1.0 \qquad (14a)$$

P_{ip} in-plane (IP) axial strength with out-of-plane (OOP) force,

P_{ipo} in-plane (IP) axial strength without out-of-plane (OOP) force

M_{oop} out-of-plane bending strength with in-plane force

M_{oopo} out-of-plane bending strength without in-plane force

Al-Chaar (2002) provides a simplified method to account for interaction using reduction factors, rather than by analysis. Al-Chaar (2002) suggests that in-plane capacity be reduced if the out-of-plane forces are more than 20% of the out-of-plane capacity, according to the following equation.

$$\frac{IP_{reduced}}{IP_{capacity}} = 1 + \frac{1}{4}\frac{OP_{demand}}{OP_{capacity}} - \frac{5}{4}\left(\frac{OP_{demand}}{OP_{capacity}}\right)^2 \qquad (15a)$$

where:

$IP_{reduced}$ in-plane capacity considering out-of-plane loading

$IP_{capacity}$ in-plane capacity found from the section on general procedures for evaluating capacity of infilled frames using pushover analyses

OP_{demand} out-of-plane demand placed on the infilled frame

$OP_{capacity}$ out-of-plane capacity found from the section on out-of-plane strength evaluation.

Recent work (Mosalam and Gunay, 2014) shows experimental data that agrees with the Kadysiewski and Mosalam equation, but designers can use either equation.

Preventing Column Shear Failure

As infill panels become stronger, the shear forces that they can transfer to the adjacent columns become larger. Column shear failure caused by interaction with infills is very undesirable; it is a commonly observed failure mechanism in traditional infilled frame buildings (Figure 8). This failure mechanism can be prevented by employing capacity design principles, where the shear strength of the framing members surrounding the infill exceeds the sum of (1) the maximum force that can be delivered by the infill panels and (2) the shear force due to the column end moments. Shear forces from the diagonal strut must be considered when designing the columns for shear using conventional capacity design.

The following equation gives the shear demand in a particular column, considering strut forces:

$$V_{col} = V_{ine} + V_p = A_{ni} f_{vie} + 2M_p/\mathrm{L} \qquad (16a)$$

Because the objective is preventing column shear failure, the expected, unreduced strength for the infill should be used to check column shear. In the equation above, V_{ine} is the horizontal component of the equivalent strut force previously described, M_p is the plastic moment capacity of the column, and L the effective length (effective height) of the column. For solid panels, the effective length can be taken as the distance between the plastic hinges, while for panels with openings, the effective length can be taken as the clear height of openings in the wall.

3.3 Design example showing concepts and requirements

Building Configuration and Details

The plan view of a four-story, residential reinforced concrete building is shown in Figure 27. Beams are 305 mm x 457 mm (12”x18”), and columns are 457 mm x 457 mm (18”x18”). The building has solid clay brick infill walls. For the purposes of this example, details of the reinforcing steel in the beams and columns are not necessary. The elevation view of Frame A is shown in Figure 28. This example checks the strength capacity of the infill in Frame A for seismic loading in the longitudinal direction using linear analysis of an equivalent strut model.

Because the factors used to determine design seismic demands and capacities vary by country and are tuned to local practice, this example contains a more general procedure. This example uses unreduced base shear in the earthquake for which collapse is unacceptable (often the Maximum Considered Earthquake, or MCE), as well as expected unreduced strengths without strength reduction factors, instead of nominal design values. Recognizing that some damage to unreinforced infill panels will occur before complete failure, a demand-to-capacity ratio of up to three is permissible, assuming good quality masonry construction. This demand-to-capacity ratio is based on lower bound infill wall component demand modification factors (“m-factors”) from Table 7-8 of a commonly referenced and freely available standard for performance based evaluation of buildings, FEMA 356 (this table becomes Table 11-8 in the later document ASCE-41-13).

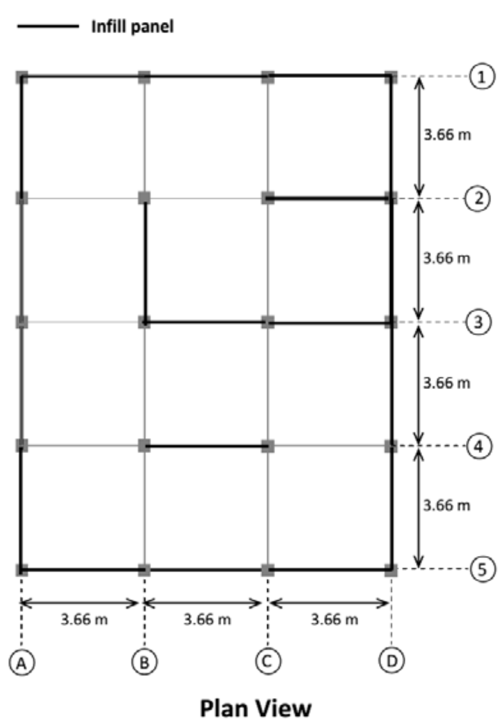


Figure 27. Plan view of the four-story residential building

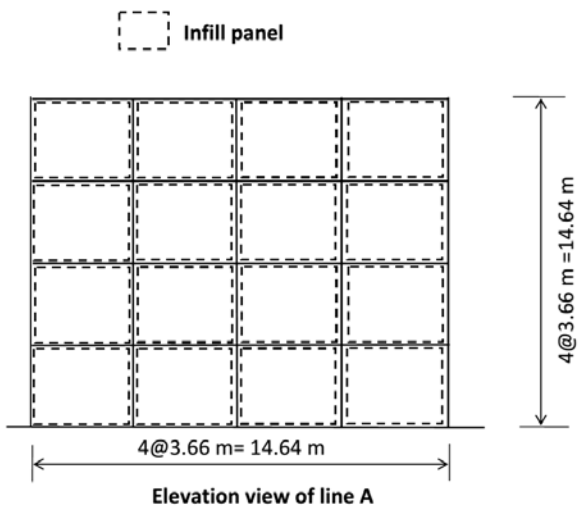


Figure 28. Elevation view of Frame A of the four-story building

The seismic weight of the building (including the wall weight and finishes) is as follows:

Seismic weight of roof: $1776\ kN$

Seismic weight of other floors: $2424\ kN$

Total seismic weight: $W = 9048\ kN$

Material Properties

It is recommended that the typical properties of masonry in local codes and/or databases be used for design purposes. In this example, the infill panels are solid brick masonry with the following properties:

Masonry weight: $w_{masonry} = 20.42\ kN/m^3$

Compressive strength of masonry: $f'_m = 8.51\ MPa$

Elastic modulus of masonry: $E_m = 550 f'_m = 3603\ MPa$

Shear modulus of masonry: $E_v = 0.4 E_m = 1441\ MPa$

Elastic modulus of concrete: $E_{bc} = 24856.48\ MPa$

Reinforced concrete weight: $w_{concrete} = 22.78\ kN/m^3$

Exterior Infill thickness: $t_{inf} = 235\ mm$

Infill height: $h_{inf} = 3.66\ m - column\ depth = 3200\ mm$

Strength of the Equivalent Strut Model

According to the procedure described in this section, the equivalent strut parameters are calculated as:

$$L_{diag} = 4526\ mm$$

$$w_{inf} = 494\ mm$$

$$Strut\ angle\ \theta = 0.79\ radians$$

The bearing (compressive strength) of the masonry infill is calculated as:

$$F_{mc} = f'_{m}\left(\frac{h_{inf}}{3}\right) \times t_{inf} = 2134\ kN$$

The shear strength of the masonry infill is calculated by first determining V_n , which is taken as the smallest of:

(a) $0.33 A_n \sqrt{f'_m}$ =724 kN

(b) $0.83 A_n$ =624 kN

$V_{n,\inf}$ is the smallest of (a), (b) and (c) as follows:

(a) $(150\,mm)\, t_{net,\inf}\ f'_m$ =300 kN

(b) The calculated horizontal component of the force in the equivalent strut at a horizontal racking displacement of 25 mm.

= $k_{inf} \times displacement\ of\ 25\ mm = \frac{E_m w_{inf}\, t_{inf}}{L_{diag}} (\cos(\theta_{strut}))^2\ (25\ mm)$ =1428 kN

(c) $V_n/1.5$= 416 kN

Thus, $V_{n,inf} = 300\ kN$ and the corresponding equivalent strut strength is derived as

$$V_{mc} = \frac{300\ kN}{\cos\theta} = 424\ kN$$

The strength of the equivalent strut is taken as the minimum of the bearing strength and the shear strength:

$$min(V_{mc}, F_{mc}) = 424\ kN$$

Calculation of Seismic Demands

The building in this example is located on stiff soil with 183 m/s < Vs < 366 m/s (V_s is the top 30 m soil shear wave velocity).[6] There are several equations that can be used to estimate the fundamental period of the example structure. Using one approximate equation, T $= 0.1\ N$, in which N is the number of

[6] This type of soil corresponds to NEHRP site class D, Eurocode 8 class C, Indian code (IS 1893:2002) Type II, and Uniform Building Code class S_D.

stories, the fundamental period of the structure is estimated as 0.4 s. Other equations are also suggested in different codes. One popular approximation for the fundamental period of the structure is $T = C_t {h_n}^x$, where $h_n$is the height of the building and C_t and x are variables that depend on the structural system of the building.

In the design example, the height of the building is $h_n = 14.64\ m$. The Indian code (IS 1893:2002) recommends that $x = 0.75$ and $C_t = 0.075$, which results T=0.56. According to Eurocode 8, $x = 0.75$ and $C_t = 0.05$, which results in T=0.37 s. According to ASCE 7-10, $x = 0.75$ and $C_t = 0.0488$, which gives T=0.36 s. To be conservative, the lowest estimated fundamental period of T=0.36 s is used in this example.

Many building codes determine the seismic design forces using a procedure that is similar to the one that is used here. The design seismic base shear is calculated using the following equation:

$$V_b = C_s W$$

in which C_s is the seismic design coefficient and W is the effective seismic weight of the structure.

$$\mathrm{C_s} = \frac{\mathrm{F_a\ S_a/g}}{\mathrm{(R/I)}}$$

R is the response reduction factor, which depends on the expected seismic performance of the lateral-force-resisting system of the structure. This factor is determined according to the level of ductility or brittleness in the behavior of the structure. I denotes the importance factor, which depends on the functional use of the structure. Important buildings have higher importance factors, which are intended to reduce damage.

$\mathrm{S_a}$ is the design spectral acceleration parameter for the Maximum Considered Earthquake (MCE), which is determined using response spectra for the site conditions of the building. $\mathrm{S_a}$ is then modified by the factor F_a to account for site conditions. A typical response spectrum is shown in Figure 29.

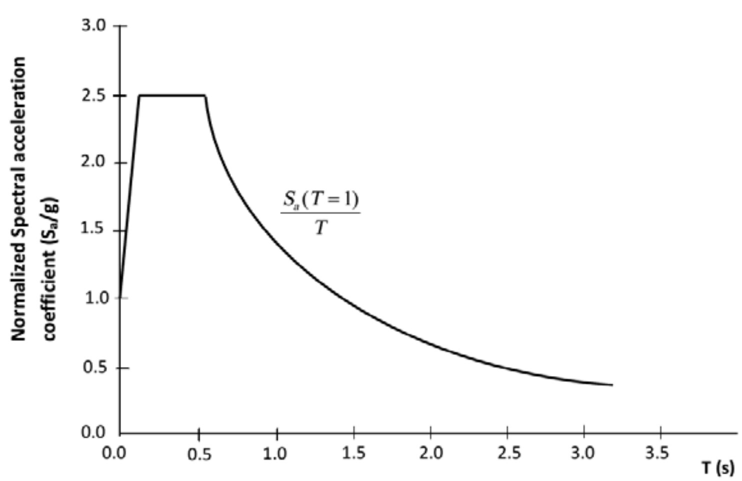


Figure 29. Typical normalized response spectrum

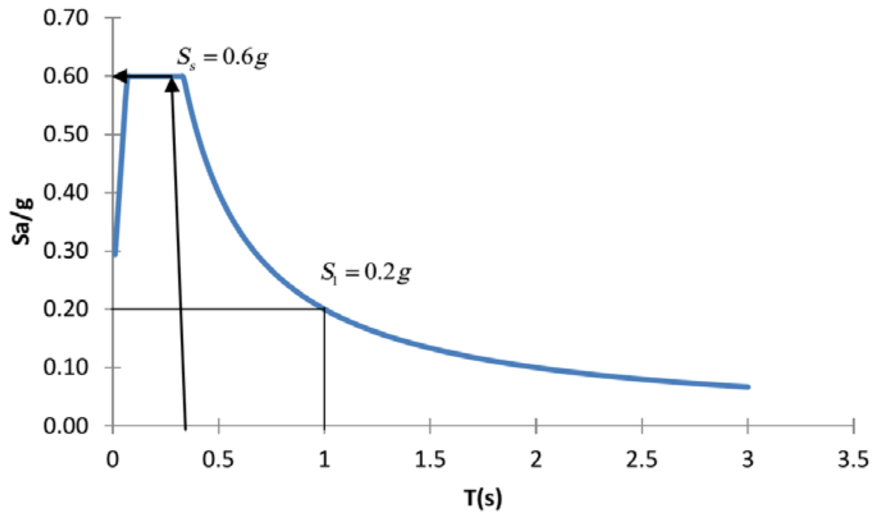


Figure 30. Normalized response spectrum

For the soil conditions specified in this example, $F_a = 1.34$ is selected. In this example, the importance factor, I, is taken as equal to 1. The unreduced base shear, V_b, is used to calculate the required strength of the infill panels, so R is set equal to 1. The reduced base shear with the response reduction factor R=3 should be used to design the building's concrete frame, assuming the ductile reinforcement detailing requirements for an ordinary moment-resisting frame are met. Accordingly, the seismic response coefficient used to determine infill panel strength is

$$C_s = 0.804$$

Therefore, the unreduced seismic base shear (with R=1) is:

$$V_b = C_s W = 7274\ kN$$

The seismic base shear is distributed along the height in accordance with the following equation:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where h_x is the height from the base to level x. Different codes recommend different values for the exponent k. For example, The Indian code (2002) recommends that k=2, and Eurocode 8 recommends that k=1. US code ASCE 7-10 recommends using k=1 for structures having a period of 0.5 s or less, using k=2 for structures having a period of 2.5 or more, and k is determined by linear interpolation between 1 and 2 for structures with a period between 0.5 and 2.5. Since the fundamental period of the structure in this example is T=0.36 s, k=1 is chosen to distribute the forces along the height. The lateral load distribution is summarized in Table 5.

Table 5. Distribution of the equivalent lateral load along the building height

Floor	Weight (kN)	Floor height (m)	$w.h^k$ (with k=1)	C_{vx}	F_x (kN)
Roof	1776	14.64	26004	0.328	**2387**
4	2424	10.98	26615	0.336	**2443**
3	2424	7.32	17743	0.224	**1629**
2	2424	3.66	8872	0.112	**814**

This equivalent linear force procedure for calculating the seismic demands is adopted by many codes and guidelines, with some differences in the modification factors applied to derive the seismic base shear. Designers should follow the provisions specified by relevant local codes in order to determine the seismic demands, but when following this design example should choose the level of earthquake for which they would like to prevent collapse, which is generally the Maximum Considered Earthquake.

The design forces in the equivalent struts can be determined from an elastic analysis of a braced frame model that includes the equivalent struts acting in compression only. For the purposes of designing the infill panels, we will assume that only the panels in frames A and D will contribute to lateral resistance, since the interior infill panels may be removed or relocated after construction. (However, when designing the reinforced concrete frames, the effects of the infill panels on the interior frames should be considered.). From linear elastic analysis, the forces acting at each level for each frame, and the resulting strut forces, are:

Table 6. Design forces in the struts

Floor	Story Lateral Loading (kN) for each frame	Infill Design Strut Force $P_{inf,\ design}$ (kN) from linear elastic analysis
Roof	1194	400
4	1222	681
3	815	881
2	407	861

Capacity check

The capacity of the masonry infill panels is determined using expected strengths and with strength reduction factor ϕ =1.0. A demand-to-capacity ratio of 3 is allowed for the panels, based on lower-bound component demand modification factors (“m-factors”) for unreinforced infill panels from FEMA 356. The capacity of the masonry infill panels is checked by:

$$\max(P_{inf,design}) = 881\ kN \leq 3\ \Phi \times (strength\ of\ equivalent\ strut) = 3(1.0) \times\ 424\ kN$$
$$= 1273\ kN$$

Therefore the panels are strong enough.

When designing the concrete frame, the designer must check that the columns and beam-column joints have sufficient shear strength to resist the forces from the equivalent struts calculated above, in order to prevent a shear failure. This example checks panel strength but does not include a check of column and beam-column joints or a check of lateral deformation of the infill panels.

4 Design Strategy B: Rocking Spine

4.1 Motivation and Design Rationale

Research and field investigations following earthquakes have demonstrated the benefits of rocking behavior in reducing force and deformation demands in a structure. In recent years, a number of rocking systems have been developed for earthquake-resilient buildings and bridges that facilitate quick and economical post-earthquake repairs. These systems typically involve the use of ductile energy dissipation devices that would be cost-prohibitive in many parts of the world, particularly for residential infill buildings. In this section, a new approach to designing and constructing infill buildings seeks to leverage the advantages of rocking behavior by making minimal modifications to the current mode of construction and thereby providing a cost-effective means of achieving an acceptable margin of safety against collapse.

In this strategy, a strong, stiff structural spine is introduced that resists earthquakes through rocking action. The spines can be constructed as slender, stout infill frames or reinforced concrete walls with shallow foundations. The use of rocking action as the primary yielding mechanism significantly reduces the required level of detailing that is needed to achieve ductility in concrete frames, resulting in significant material cost savings. The system relies on gravity and the restraint provided by structural members connected to the spine as the primary sources of overturning resistance. These include the beam elements framing into the spine as well as infill panels constructed in the adjacent bays on either side of the spine.

The primary behavioral goal of the rocking spine is to impose uniform deformations over the height of the structure. This reduces the tendency for drift demands to concentrate at the lower levels of traditional infilled frames. It also redistributes the yielding that would typically occur in the lower level columns to the adjacent beams and infill throughout the height of the structure. Cost savings are realized because of (1) the lower required level of ductility in the concrete frame, (2) a smaller architectural footprint of the spine, which can be constructed as a slender wall, and (3) the elimination of deep foundations which are typically needed for traditional mid- and high-rise shear wall structures.

4.2 Concepts and Implementation Strategies

System Description and Behavior

A schematic representation of the rocking spine system is shown in Figure 31. The spine consists of a strong, stiff infill frame with a shallow foundation and is shown centered between two adjacent, traditional infill frames. The terms “spine infill” and “non-spine infill” will be used throughout this document to distinguish between the infill panels that are part of the spine and those that are located within frames outside the spine. The strength and stiffness of the spine infill is critical to achieving the desired system performance. Ideally the spine infill and framing members are to remain elastic when subjected to low and moderate earthquakes. At larger intensities, a nominal level of damage to the spine can be accommodated based on the desired performance. The rocking spine derives all of its overturning resistance from gravity loads and the adjacent infill panels and beams that frame into it. The non-spine infill and beams in the adjacent frame on the uplift-side of the spine serve as outriggers, transferring additional gravity loads to the spine and adding to its overturning resistance. The magnitude of gravity loads transferred to the spine is limited by the strength of these outrigger elements. On the compression-side of the spine, the adjacent elements also provide overturning resistance through compatibility and their constitutive relationships. The non-spine infill and adjacent beams also serve as yielding elements and are relied on to dissipate energy under cyclic loading. Grade beams are used to connect the footings at the base of the spine to the adjacent frames to facilitate the transfer of lateral forces at the foundation after spine uplift has occurred.

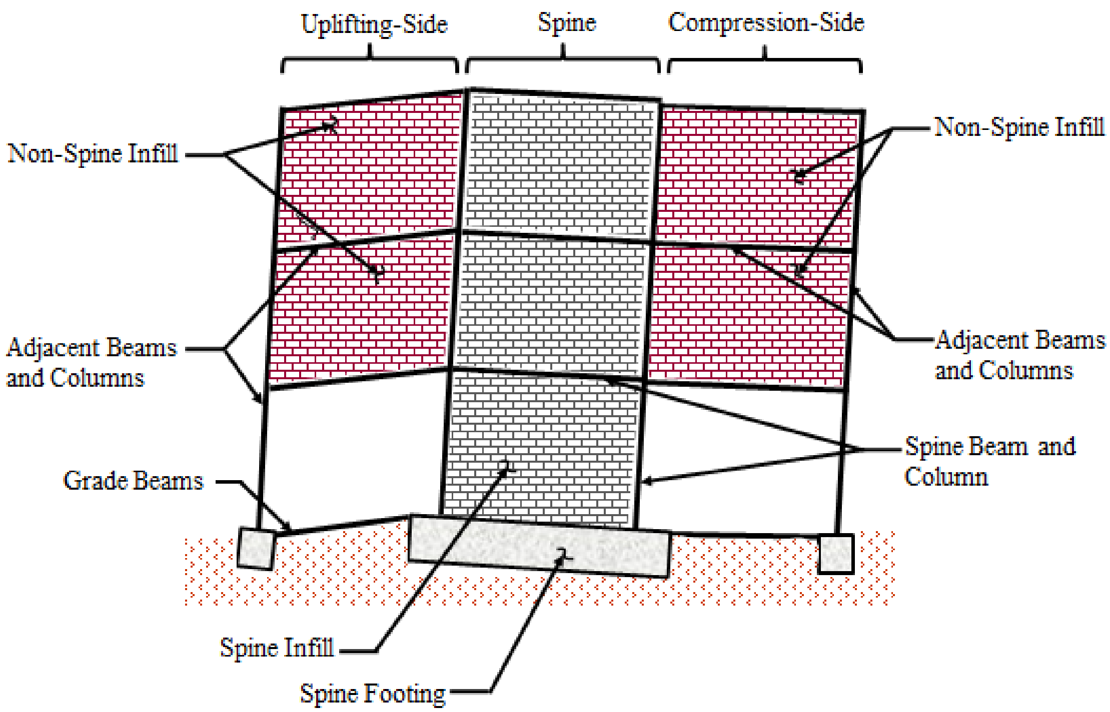


Figure 31. Rocking spine system concept

Figure 32 shows a loading diagram for the rocking spine system. The infill panels in both the rocking spine and the adjacent frames are idealized as diagonal compression struts. This is one of a number of alternative configurations that can be implemented. The behavior of the spine is significantly influenced by the absence or presence of adjacent non-spine infill (discussed later). At low levels of lateral load, the beams and columns within the spine and adjacent frames undergo elastic deformations with some minor cracking occurring in the infill panels. At higher levels of lateral loading, the overturning moment on the spine exceeds that of the resistance provided by gravity loads resulting in uplift at the footing as shown in Figure 33. Deflection in the overall system after uplift consists of elastic deformation of the framing members and infill panels and rigid-body rotation of the spine. The deflection of the spine can be described by the angle formed between the rotated footing base and the horizontal plane (θ_{up}). The horizontal (Δ_R) and vertical (Δ_V) displacements due to rigid body rotation of the spine can be calculated by assuming small angles and neglecting the elastic deformations that occur prior to uplift.

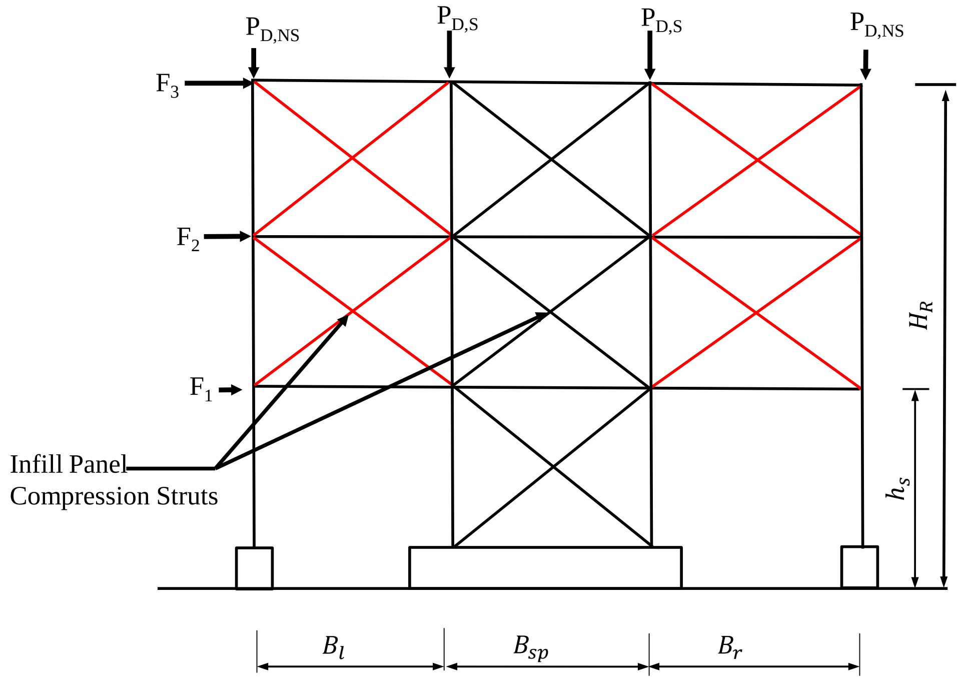


Figure 32. Idealized loading diagram for rocking spine system

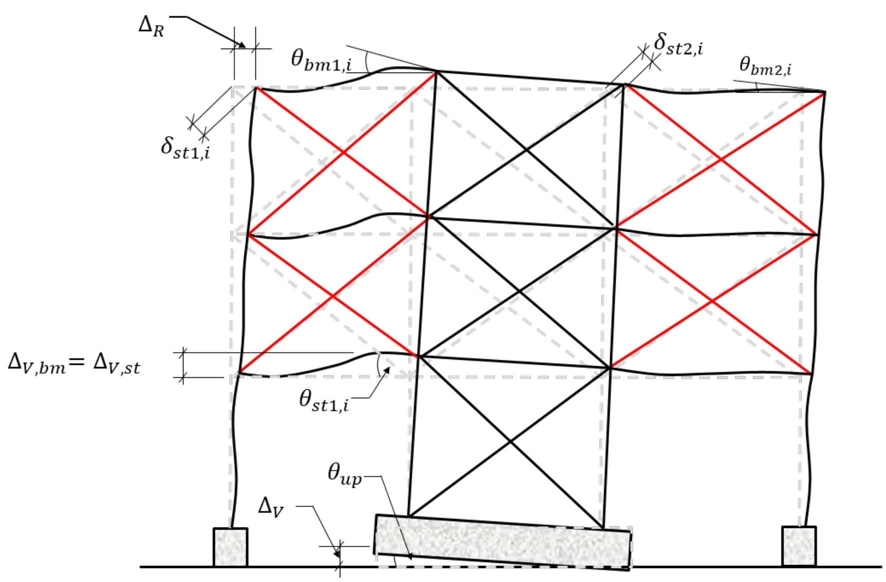


Figure 33. Deflected shape of rocking spine system after uplift

The deformation demands in the adjacent frames can be assessed based on their compatibility with the spine uplift. The lateral deflection in the adjacent frames will be the same as that of the rocking spine. The rotation demands in the adjacent beams framing into either side of the spine after uplift include the rotation due to elastic deformation of the spine plus the rotation due to spine rigid body motion. The beam on the uplift-side of the spine also undergoes a vertical translation ($\Delta_{V,bm}$) at the joint where it frames into the spine. Due to the flexibility and distribution of yielding of the framing elements, there will be some variation in the vertical translation of the adjacent beams at different story levels along the height of the building, but this is considered negligible. The magnitude of this vertical translation is assumed to be the same as the vertical displacement at the spine footing. At any given drift demand, the rotation in the adjacent beams on the compression-side can be estimated as the roof drift ratio or uplift angle of the spine.

$$\theta_{bm2} = \theta_{roof} \tag{1b}$$

where θ_{bm2}: rotation in the adjacent beam on the compression-side of spine

The uplift-side beams are subjected to rotational demands both as a result of spine lateral drift as well as vertical translation.

$$\theta_{bm1} = \theta_{bm1,1} + \theta_{bm1,2} \tag{2b}$$

where θ_{bm1} : total rotation in the adjacent beam on the uplift-side of spine

$\theta_{bm1,1}$: rotation in the adjacent beam on the uplift-side of spine due to spine lateral drift

$\theta_{bm1,2}$: rotation in the adjacent beam on the uplift-side of spine due to spine uplift

The rotation in the uplift-side adjacent beam due to spine lateral drift can be estimated as the roof drift ratio or uplift angle of the spine.

$$\theta_{bm1,1} = \theta_R \tag{3b}$$

The rotation in the uplift-side adjacent beam due to spine uplift can be computed from the resulting vertical translation in the beam

$$\Delta_{V,bm} = \theta_R B_{ot} \tag{4b}$$

where $\Delta_{V,bm}$: vertical translation in the adjacent beam on the uplift-side of spine

The resulting chord rotation can be estimated as the ratio between the vertical translation at the end framing into the spine and the length of the beam. This estimate is based on a small angle assumption

$$\theta_{1,2} = \frac{\theta_R B_{ot}}{L_{bm1}} \tag{5b}$$

where L_{bm1} : length of adjacent uplifting-side beams

Equations 1b through 5b can be combined to give the total rotation in the uplift-side adjacent beam resulting from lateral drift and spine uplift

$$\theta_{bm1} = \theta_R \left(1 + \frac{B_{ot}}{L_{bm1}}\right) \tag{6b}$$

The moment demand in the adjacent beams at any spine roof drift level can also be computed using equations (1b) through (5b), combined with their moment-rotation relationship.

The deformation demands in the infill struts can also be assessed using compatibility. The infill struts on either side of the spine will undergo axial shortening as a result of spine deflection. The infill strut on the uplift-side of the spine will undergo an additional axial shortening due to the vertical translation ($\Delta_{V,st}$) at the joint where the strut frames into the spine. As was the case with the adjacent beam, the magnitude of this vertical translation at all story levels is assumed to be the same as the vertical displacement at the spine footing. The axial shortening in the non-spine infill struts can be described using the following equations.

$$\delta_{st1,i} \; = \; \Delta_{story,i} \cos(\theta_{st1,i}) + \Delta_{V,st} \sin(\theta_{st1,i}) \tag{7b}$$

$$\delta_{st2,i} \; = \; \Delta_{story,i} \cos(\theta_{st2,i}) \tag{8b}$$

where $\delta_{st1,i}$: axial shortening of infill strut on uplift-side of spine at story *i*

$\delta_{st2,i}$: axial shortening of infill strut on compression-side of spine at story *i*

$\theta_{st1,i}$: uplift-side infill strut angle at story i

$\theta_{st2,i}$: compression-side infill strut angle at story *i*

$\Delta_{story,i}$: interstory displacement at story *i*

As was the case with the adjacent beams, the force demands in the adjacent non-spine infill can be computed from equations (7b) and (8b) combined with their force-deformation relationship.

In cases where the response of the rocking spine is dominated by rigid body rotation, the displacements due to elastic deformations can be ignored, which is a reasonable assumption for low and mid-rise frames.

Figure 34 shows a free body diagram of the spine after it has experienced uplift, highlighting the sources of overturning resistance. The gravity loads acting directly on the spine ($P_{grav,sp}$) provide a restoring moment. Gravity loads on the column one bay over from the uplift-side of the spine ($P_{grav,ns}$) are transmitted to the spine through outrigger action of the adjacent beams and non-spine infill. The uplift-side adjacent beams provide overturning resistance from its end moment (M_{bm1}) and the restoring moment from its end shear force (V_{bm1}). The compression-side adjacent beams provide overturning resistance from its end moment (M_{bm2}). The uplift-side, non-spine infill generates a restoring moment from the vertical component of its strut force (P_{st1}). However, other non-spine infill (compression-side and non-adjacent) do not significantly influence the overturning resistance of the spine. However, they do influence the distribution of forces along the height of the spine and as a result, the maximum shear that is delivered to the spine.

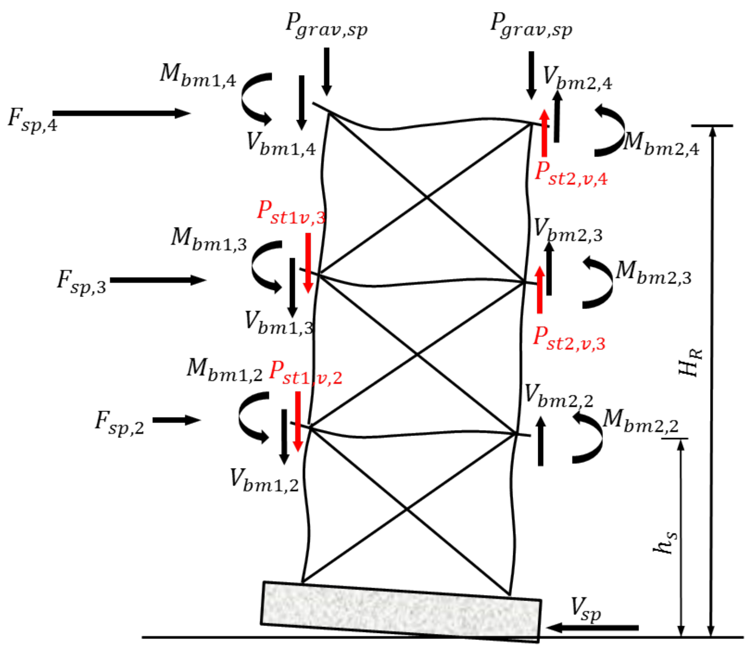


Figure 34. Free-body diagram of spine after uplift

4.3 Possible Design Approaches

There are three main approaches that could be adopted to incorporate a rocking spine system:

1. Using a rocking spine as the primary lateral-load-resisting system. In this case, frames are designed only for gravity loads and deflection compatibility, which results in cost savings.
2. Using spine and moment frames as lateral load resisting systems in order to obtain additional strength.

3. Using spine, moment frame, and ductile infill. In this case, additional strength is obtained by adequate detailing of infills.

The methodology for implementing the first approach is described in this document. Further detailing and provisions should be made to accommodate the other two approaches.

System and Component Limit States

Figure 35 shows the system and component load-deflection curve for the rocking spine system with and without infill frames on either side. The load deflection relationship for the rocking spine system is expressed in terms of overturning moment and uplift. After uplift has occurred, the overturning moment can be calculated from the gravity loads on the rocking spine, its vertical displacement, and the constitutive relationships for the adjacent beams and infill struts. The behavior of the system can be described in terms of the superposition of strength and restoring actions provided by the gravity loads, adjacent beams, and infill panels where present. Figure 35 shows the idealized pushover curve for the rocking spine system with adjacent infill panels. Figure 36 shows the idealized pushover curve for the rocking spine system without adjacent infill panels. The presence of infill panels adjacent to the spine significantly changes the pushover response and has considerable implications in the design procedures that will be presented later in the report. The relative contribution of overturning resistance from the different sources can be controlled by the strength and stiffness of the adjacent beam and infill. The designer will have less control over the magnitude of the gravity load on the spine. There will also be a restoring moment provided by the flexural resistance of the slab. This can be incorporated by utilizing T-Beam properties for the adjacent beams.

Both the adjacent beams and non-spine infills are expected to exhibit inelastic response, thereby providing energy dissipation through their hysteretic response. The non-spine infill panels are expected to exhibit a brittle response and contribute significantly to the overturning resistance at very low drift levels. The adjacent beams framing into the spine are expected to exhibit a more ductile response than the infill panels but with less contribution to the overturning resistance. The level of ductility in the adjacent beam response will be governed by the detailing of the beam-column connection at the spine. The detailing requirements for this connection will be discussed later in the report.

The limit states of the rocking spine system with adjacent infill panels are shown in Figure 35. Prior to uplift, the rocking spine experiences very small levels of story drift due to the elastic deformation in the infill panels (minor cracking in the infill is also expected during this stage) and framing members. When the overturning moment exceeds the restoring moment, uplift occurs at the base of the spine footing. As the vertical and lateral deflection of the rocking spine increases, the adjacent beams and infill panels undergo increased deformations, leading to the onset of significant cracking in the adjacent infill panels. Recall that the infill panels on the uplift-side of the spine experience deformations due to both lateral drift and uplift of the spine, while the non-spine infills on the compression-side only experience deformations due to lateral drift. As a result, at any given point on the pushover curve, the non-spine panels on the uplift-side are expected to experience greater levels of damage than those on the compression side.

The onset of strength loss in the adjacent infill panels also coincides with the onset of strength loss in the rocking spine system. Repair of the rocking spine system up to this point will likely involve restoration of the adjacent infill panels. As deformations increase beyond this limit state, the non-spine infill panels continue to degrade until they are no longer able to contribute to the restoring moment in the rocking spine. The onset of yielding in the adjacent beams, particularly on the uplift-side of the spine, is likely to take place during the degradation of the adjacent infill panels. After the complete degradation of the adjacent infill panels, the adjacent beams continue to undergo inelastic deformations. The onset of strength degradation in the adjacent beams represents a critical limit state which will lead to significant loss of strength and stiffness of the rocking spine system. Therefore the limit state of the onset of strength loss in the adjacent beams should be considered a life safety threat. With increased drift demands, the adjacent beams will continue to degrade to the point of complete strength loss. At this point, the gravity loads on the spine becomes the last layer of protection against excessive rocking and overturning of the rocking spine. For the rocking spine without adjacent infill panels, the system backbone curve is governed by the inelastic deformations in the adjacent beams.

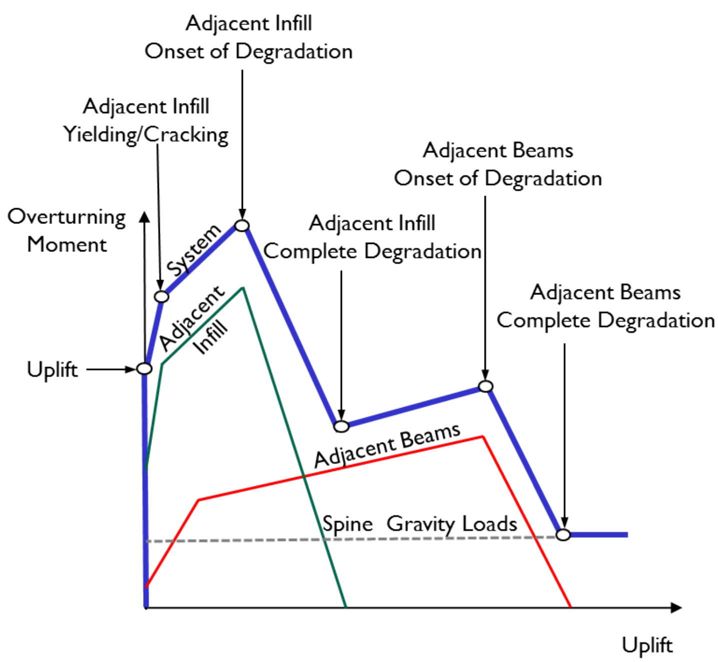
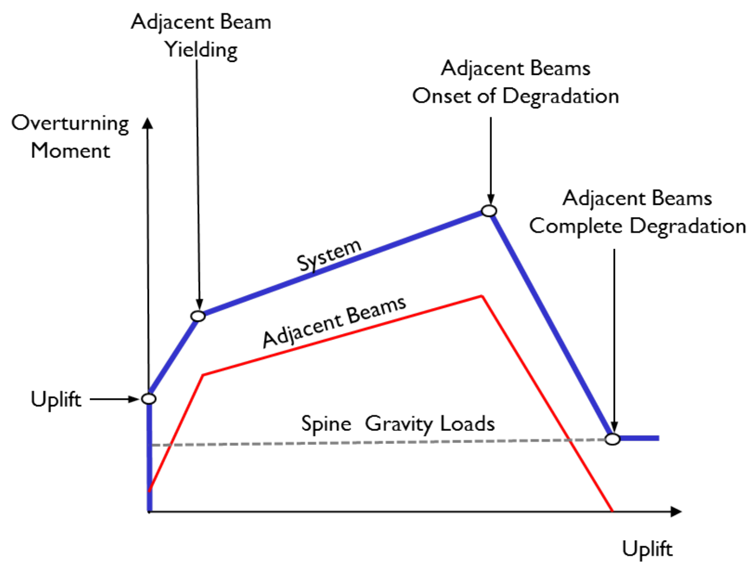


Figure 35. Idealized pushover curve and limit states for rocking spine with adjacent infill panels



(b)

Figure 36. Idealized pushover curve and limit states for rocking spine without adjacent infill panels

Design Considerations

The design process begins with the assumption that the building geometry and seismic weights are known. With these two pieces of information in mind, the seismic design process consists of the following key steps:

1. Determine the design base shear and overturning moment
2. Allocate the rocking spine system's overturning resistance among gravity restoring forces, the adjacent beams framing into the spine, and infill panels (where present) adjacent to the spine
3. Compute drift demands and check them against the proposed limits
4. Determine design forces for structural components comprising the spine
5. Design the following key structural components that comprise the rocking spine system:
   - The infill panels that comprise the spine
   - The adjacent infill panels (where present) that help resist the design seismic load
   - The beams and columns within the spine
   - The beams and columns adjacent to the spine that help resist the design seismic load

Design Base Shear and Overturning Moment

The base shear and overturning moment can be determined using the equivalent lateral force method as described in most national building codes. The following steps are needed to compute the base shear and overturning moment using the force-based methodology:

1. An approximate fundamental period is computed for the building with the rocking spine system classified as a wall structure.

2. The design spectral acceleration parameters are computed based on the site seismicity.

3. The design base shear is computed using the equivalent lateral force (ELF) procedure assuming an appropriate seismic response modification factor (R-factor). (Using an R-factor between 4 and 6 is probably reasonable for most systems.)

4. The design base shear is distributed along the building height in accordance with the ELF procedure.

5. The design overturning moment is computed based on the vertically distributed design lateral forces.

Once the lateral load distribution is obtained, the overturning moment is computed using

$$M_{ot,u} = \sum_{i=1}^{n_{levels}} F_i h_i \tag{9b}$$

where $M_{ot,u}$: is the design overturning moment

F_i: is the equivalent lateral force at level *i*

h_i: is the height of level *i* above the ground floor

Allocation of Rocking Spine System Strength

The lateral strength of the spine is described in the form of an overturning resistance that is derived from the following three potential sources:

1. Gravity loads on the spine
2. Adjacent beams that frame into the spine
3. Infill panels adjacent to the spine.

The adjacent beams and infill panels act as outriggers transferring gravity loads from adjacent bays. The configuration of the rocking spine within the overall building system is such that sources (1) and (2) are always present, since there will always be adjacent connecting beams and some level of gravity load on the spine. The designer has some control over the magnitude of overturning resistance provided by these two sources. The resistance provided by the adjacent beams is controlled through their flexural strength. The magnitude of gravity loads on the spine can be nominally controlled through its location within the overall building system and the layout of framing members in its vicinity. The location of non-spine infill panels within the building system will typically be controlled by architectural or functional constraints. While the adjacent infill panels can be a source of significant overturning resistance, their presence can greatly increase the demands on the structural components that encompass the spine.

The presence of infill panels adjacent to the spine also dramatically changes the characteristics of the pushover curve and overall yielding mechanism of the system, as was shown in Figure 35. With these considerations in mind, two design approaches are developed that incorporate the presence or absence of infill panels adjacent to the spine.

Allocation of Rocking Spine System Strength in the Absence of Adjacent Infill

The previous section outlined the procedures for obtaining the design overturning moment for the system ($M_{ot,u}$). In the absence of adjacent non-spine infill, this design overturning moment is resisted by restoring moments from the dead load on the spine and beams that frame into the spine. Figure 34 shows a free body diagram of the spine with the lateral forces that generate the design moment in addition to those that provide overturning resistance, which in this case include the gravity loads on the spines and end reactions from adjacent beams. Given the required design overturning moment, the nominal overturning moment strength ($M_{ot,n}$) that is required from the two available sources can be calculated from the following inequality.

$$\phi M_{ot,n} \geq M_{ot,u} \tag{10b}$$

where ϕ: resistance factor = 0.9

$M_{ot,u}$: design overturning moment

$M_{ot,n}$: nominal overturning moment resistance

The nominal overturning moment resistance can be calculated from the following equation.

$$M_{ot,n} = \frac{P_D B'_{ot}}{2} + n_{bm} M_{n,bm} + m_{bm} V_{n,bm} L_{spine} \tag{11b}$$

where P_D: total dead load on spine

B'_{ot}: distance between tension column and centroid of soil compression block

$M_{n,bm}$: nominal flexural strength of adjacent beams

$V_{n,bm}$: nominal shear strength of adjacent beams

n_{bm}: number of beams framing into spine

m_{bm}: smaller of the number of in-plane and out-of-plane beams framing into either end of the spine (e.g. if 6 beams frame into one end and 4 into the other end, m_{bm} = 4)

Assuming that the adjacent beams will be designed as flexure-controlled elements, the shear force corresponding to their plastic moment capacity can be computed using the following equation.

$$V_{n,bm} = \frac{2M_{n,bm}}{L_{bm}} \tag{12b}$$

The required nominal flexural strength of the adjacent beams can then be computed using the following relationship.

$$M_{n,bm} = \frac{M_{ot,n} - \frac{P_D B'_{ot}}{2}}{\left(n + \frac{2mB'_{ot}}{L_{bm}}\right)} \tag{13b}$$

Allocation of Rocking Spine System Strength in the Presence of Adjacent Infill

In the case where spines are located in the presence of adjacent infill, there is an additional contribution to overturning resistance provided by the uplift-side, non-spine infill that can be computed using the following relationship.

$$M_{n,inf} = n_{inf} F_{u,inf} sin(\theta_{inf}) B'_{ot} \tag{14b}$$

where n_{inf} : smaller of the number of adjacent infill panels on either end of the spine (e.g. if there are 6 adjacent panels on one end and 4 on the other end, n_{inf} = 4)

$F_{u,inf}$: Ultimate strength of equivalent infill strut

θ_{inf} : Angle between equivalent infill strut and horizontal plane

The total nominal overturning moment resistance of the rocking spine system can then be calculated from the following equation.

$$M_{ot,n} = \frac{P_D B'_{ot}}{2} + n_{bm} M_{n,bm} + m_{bm} V_{n,bm} B'_{ot} + n_{inf} F_{u,inf} sin(\theta_{inf}) B'_{ot} \tag{15b}$$

The strength of the non-spine infill is likely to be pre-determined based on the size and strength of masonry units used to construct the panel. The bond strength between the masonry joints, the axial load on the infill wall, and the diagonal strut force will also affect the maximum strength that can be developed. However, the strength of the non-spine infill panels should be limited to "low" or "moderate" levels such that the required strength of the spine does not exceed that which can be realistically constructed from the available materials. Given the strength of the non-spine infill panels and the magnitude of dead load on the spine, the required strength of the adjacent beams can be computed as follows.

$$M_{n,bm} = \frac{M_{ot,n} - \frac{P_D B'_{ot}}{2} - n_{inf} F_{u,inf} sin(\theta_{inf}) B'_{ot}}{\left(n + \frac{2mB'_{ot}}{L_{bm}}\right)} \tag{16b}$$

In some cases, the strength of the non-spine infill panels will be strong enough such that the strength of the adjacent beams needed for overturning resistance is less than that which is needed for gravity. In that case, the design of the adjacent beams will be controlled by gravity. Recall that the overturning resistance generated through outrigger action of adjacent beams and infill panels cannot be greater than the product of the total gravity acting on the column one bay over from the uplift-side of the spine times the width of the spine.

Computing Drift Demands

The maximum drift demand on the structure can be assessed using the methodology developed by Ma et al (2011) for the rocking frame. The approach employs some aspects of the direct-displacement design methodology with a few modifications and is based on an assumption of rigid body motion of the rocking system.

Predicting Force Demands on Spine Structural Components

This section presents a methodology for estimating the force demands that are used to design the structural components that comprise the spine. These design forces are highly dependent on the assumed lateral force distribution along the height. Priestly (2003) developed a modified modal superposition method for computing the design shear forces in flexure-controlled cantilevered walls, particularly those with significant contributions from higher modes. Previous studies have found that ductility primarily acts to limit force demands in the first mode response with higher modes subjected to elastic force levels. The modified modal superposition was developed to recognize this characteristic of nonlinear behavior. Modal superposition is conducted whereby the reduced first mode forces are combined with elastic higher mode force levels. A similar approach is presented for computing the force-demands on the spine; however, a few minor modifications are needed to account for the incorporation of the rocking system as the primary yielding mechanism.

The following are the major steps involved in the adapted modified modal superposition approach developed to predict design lateral forces on the rocking spine.

1. Compute the elastic modal story forces using the following relationships derived from the response spectrum analysis method (Chopra 2007).

$$f_{i,n} = \Gamma_n [m] \left\{\phi_{i,n}\right\} S_{a,n} \tag{17b}$$

$$\Gamma_n = \frac{\left\{\phi_n\right\}^T [m]\{1\}}{m_n^*} \tag{18b}$$

$$m_n^* = \left\{\phi_n\right\}^T [m]\left\{\phi_n\right\} \tag{19b}$$

where $f_{i,n}$: the *nth* mode story force at level i

Γ_n: participation factor for the *nth* mode

$[m]$: mass matrix

$\{\phi_{i,n}\}$: *nth* mode shape at level *i*

$S_{a,n}$: pseudo spectral acceleration for the *nth* mode

2. Compute the probable maximum overturning resistance for the rocking spine system, which is taken as the sum of the probable maximum overturning resistance for each of the sources of strength.

$$M_{ot,pr} = \sum M_{grav,exp} + \sum M_{bm1,pr} + \sum M_{bm2,pr} + \sum M_{st1,pr} \tag{20b}$$

$$\sum M_{grav,exp} = \frac{(1.05P_D + .25P_L)B'_{ot}}{2} \tag{21b}$$

$$\sum M_{bm1,pr} = n_{bm1}M_{pr,bm1} + n_{bm1}V_{pr,bm1}B'_{ot} \tag{22b}$$

$$\sum M_{bm2,pr} = n_{bm2}M_{pr,bm2} \tag{23b}$$

$$\sum M_{st1,pr} = n_{st1}P_{st1,exp}sin(\theta_{st})B'_{ot} \tag{24b}$$

where $M_{ot,pr}$: probable maximum overturning resistance for the rocking spine

$\sum M_{grav,exp}$: total overturning resistance from expected gravity loads (1.2DL + .25LL)

$\sum M_{bm1,pr}$: probable maximum overturning resistance provided by all uplift-side adjacent beams

$\sum M_{bm2,pr}$: probable maximum overturning resistance provided by compression-side adjacent beams

$\sum M_{st1,pr}$: probable maximum overturning resistance provided by uplift-side non-spine infill

$M_{pr,bm1}$: maximum probable moment acting at the ends of uplift-side adjacent beams

$M_{pr,bm2}$: maximum probable moment acting at the ends of compression-side adjacent beams

$V_{pr,bm1}$: shear corresponding to the probable maximum moment acting at the ends of adjacent beams plus gravity loads

$P_{st1,exp}$: expected axial strength of uplift-side infill strut

n_{bm1}: number of uplift-side beams

n_{bm2}: number of compression-side beams

n_{st1}: number of uplift-side non-spine infill panel struts framing into spine

3. Scale the elastic, first mode story forces such that the resulting overturning moment matches the maximum probable overturning resistance for the rocking spine and the following relationship holds

$$\sum_{i}^{ns} f_{i,1}^{s} H_i = M_{pr,ot} \tag{25b}$$

where $f^{s}_{i,1}$: scaled first mode story force at level *i*

H_i: height of level *i*

4. Convert elastic modal story forces to story shears using the following relationship and the scaled modal story forces from step 3 for the first mode

$$F_{i,n} = \sum_{i}^{n} f_{i,n} \tag{26b}$$

where $F_{i,n}$: *nth* mode story shear below level *i*

n: total number of stories in building

5. Combine modal story shears using the square root sum of the squares

$$F_i = \sqrt{\sum_{n=1}^{n_{modes}} F_{i,n}^2} \tag{27b}$$

where F_i: design story shear below level *i*

n_{modes}: number of modes considered in response spectrum analysis

The design forces for the structural components that make up the spine are taken as the story shears computed in step 5.

Design of Spine Structural Components

This section will address the design of the key structural components that comprise the spine including (1) the beams within and adjacent to the spine, (2) the columns within and adjacent to the spine and (3) the spine infill panels. Although the non-spine infill panels are considered in the allocation of overturning resistance of the rocking spine, there are no explicit strength requirements associated with these elements. In fact, it is recommended that they be constructed with weak material and/or with the presence of window or door openings.

Design of Beams within and Adjacent to Spine

The adjacent beams that frame into the spine are one of the sources of overturning resistance and serve as yielding elements in the rocking spine system. They are expected to undergo inelastic deformations in a major earthquake and therefore are designed as deformation-controlled elements. The design moment ($M_{u,bm}$)for the adjacent beams are taken as the larger of that which is required for overturning resistance and the moment associated with gravity loads.

$$M_{u,bm} = \max\{M_{u,ot}, M_{u,grav}\} \tag{28b}$$

where $M_{u,bm}$: design moment for adjacent beams

$M_{u,ot}$: moment demand associated with overturning resistance

$M_{u,grav}$: moment demand associated with gravity loads

Adjacent beams are to be designed as flexure-controlled elements to ensure a ductile response in the rocking spine system. Therefore the design shear forces are to be computed from statics assuming the maximum probable flexural strength (M_{pr}) acts at the joint faces and the member is loaded with factored gravity load as required in ACI 21.3.4.1. The maximum probable flexural strength is to be taken as 1.25 times the nominal flexural strength. Shear stirrups are to be sized based on the required shear strength considering the shear strength of the beam concrete and should have a minimum spacing of twelve inches on center.

Design of Spine Columns

The columns that are constructed as part of the rocking spine are to be designed as force controlled elements. The direction of flexural demands in the connecting beams is such that the moment demand in the spine column due to the vertical translation on the uplift side is zero. As such, the flexural demand in the spine columns does not control their design. The most critical loading conditions for these columns are the axial forces that result from spine uplift and the shear forces that result from interaction with the spine infill.

The design axial force for the spine columns is based on the combined effect of gravity and the maximum vertical force transmitted by adjacent beams and infill. The axial loads transmitted from the compression-side adjacent beams and non-spine infill has a net tensile effect on the spine columns, while the loads transmitted from the uplift-side have a net compressive effect. The design axial force on the spine columns are computed based only on the loads from the uplift side. This is a conservative assumption, but studies show that it provides reasonable predictions of design axial forces in the spine column.

$$P_{u,col,i} = \sum_{i=1}^{n_{stories}} \left(\sum_{sp} \left(P_{grav,exp,i} \right) + V_{pr1,i} + P_{exp1,i} \sin(\theta_{st}) \right) \quad (29b)$$

where $P_{u,col,i}$: design axial force for spine column at story i

$\sum_{sp}\left(P_{grav,exp,i}\right)$: total expected gravity on spine at story i

$V_{pr1,i}$: shear force in adjacent beam at level $i+1$ corresponding to its plastic moment capacity

$P_{exp1,i}$: expected strength of non-spine infill strut at story i

$n_{stories}$: total number of stories in building

Experience with earthquakes has shown infill panels can transfer large forces to the surrounding framing members, resulting in shear failure. This failure mechanism is particularly critical in columns, as this could lead to the loss of gravity load carrying capacity. The spine columns are to be designed to resist the shear forces that can be transmitted from the spine infill. The magnitude of this shear force is taken as a fraction of the axial force in the infill strut resolved in the web direction of the column. Chrysostomou (1991) used the principle of virtual displacements to investigate the force transfer columns from infill compression struts. Using this approach he found that (1) the magnitude of force transfer is a function of the lateral displacement of the wall, and (2) the maximum force delivered to the column is approximately 25% of the total infill strut force. Building on the work of Chrysostomou, this relationship between the infill strut force and the fraction transferred to the adjacent column has been established as a rule of thumb and used in other studies including one by El-Dakhakhni et al. (2003). Based on this assumption, the design shear force in the spine column will be computed as follows.

$$V_{u,col} = .25 P_{exp} \cos(\theta_{st}) \quad (30b)$$

where $V_{u,col}$: design shear force in spine column

P_{exp}: expected strength of spine-infill strut

The design flexural moment for the spine columns is based on the combination of gravity and seismic/rocking effects with the appropriate load factors. The moment demand due to rocking action can be computed based on the uplift angle or roof drift corresponding to the maximum credible earthquake or the design drift limit that is used for that same intensity. However, as was noted earlier, the flexural design of the spine column is not likely to control.

$$M_{E,col} = M_{col,\theta_{MCE}} \quad (31b)$$

where $M_{E,col}$: spine column moment due to seismic/rocking effects

$M_{col,\theta_{MCE}}$: moment demand in column at MCE roof drift or the design drift limit used for that same intensity.

The column flexural strength is determined for the axial force resulting in the lowest flexural strength.

Design of Columns One-Bay Over From Spine

Columns that are one bay over from the spine are subjected to flexural demands as a result of rocking action within the spine. This flexural demand can be computed using the similar relationships developed for the spine columns that link the spine uplift angle to moments. As such, equation 31b also applies to the columns one-bay over from the spine. However, the relationship between the spine uplift angle and flexural demands is different for the columns one-bay over from spine.

Non-spine columns that are part of a frame with infill panels are to be designed to resist the effects of shear and axial forces transmitted from the infill. These infill panels are expected to undergo inelastic behavior under the design basis earthquake; therefore the shear and axial forces transmitted to columns is based on their expected shear strength. The design column shear force can be computed using the relationship previously described, where:

$$V_{u,col} = .25P_{exp}\cos(\theta_{st})$$

The design shear force in non-spine columns one bay over from the spine without infill panels is based on the combination of gravity and rocking effects. The shear force that occurs as a result of rocking effects can be computed by assuming the design moment computed in equation 30b acts at the ends of the column.

The design axial force for non-spine columns is computed from the combination of gravity and rocking effects. The axial force due to rocking is based on the expected strength of the infill panels within the frame.

$$P_{u,col} = .25P_{exp}\sin(\theta_{st}) \quad (32b)$$

Where no infill panels are present, non-spine columns are designed for gravity loads only.

Design of Spine Infill Panels

The infill panels that are part of the spine are designed to remain elastic under the Design Basis Earthquake (DBE). Some level of damage is allowed at the Maximum Considered Earthquake (MCE); however, this damage should precede the onset of strength loss in the panels. The design force in the spine infill panels are computed as the axial force in the infill struts associated with the design lateral forces on the spine computed above. Recall that the design lateral forces on the spine are obtained using capacity design principles with the maximum expected overturning resistance being the limiting factor.

5 Design Strategy C: Strong but Ductile or Energy Dissipating Base

5.1 Motivation and Design Rationale

In buildings with a ground story that is either fully open or substantially more open than the stories above, it is economically attractive to add a few, strong elements in the ground story to prevent a weak-story collapse. This is the case in both new construction and retrofit. In many buildings throughout the world, the ground story is used for shops, parking, or pedestrian access, while the stories above contain residences. It is appealing to use fewer, stronger elements in the ground story to preserve larger open spaces, while relying on the much larger wall density in the residences above for lateral resistance. Residences typically need more walls than other functional spaces, in order to separate apartments and individual rooms. The design strategy in this section provides a practical and cost effective way to design a safe ground story in alignment with architectural needs. For new buildings, specific additional measures are required only in the ground story; the upper stories can be built as standard infill buildings are typically built. This strategy was originally developed for retrofits because it is usually economically necessary to minimize or eliminate the cost and disruption involved in structural work in residences.

Adding energy dissipating elements, such as buckling-restrained braces or dampers, is another way to reduce the deformations in an open ground story. This may be too costly or difficult in some countries but can be feasible in higher-income countries where infill construction is common (such as some countries in the Mediterranean region). Energy dissipating elements can be especially attractive as a retrofit because they minimize disruption to upper-floor residences and the associated costs. Also, the building's configuration and the size, strength and stiffness of infill walls relative to the frame may make it technically difficult to use reinforced concrete walls in the ground story. For adding strength and ductility, the speed and typically less intrusive construction to install energy dissipating elements such as buckling-restrained braces or dampers make them a viable alternative to reinforced concrete shear walls or specially-designed infill panels.

5.2 Concepts and Implementation Strategies

"Tuning" the Strength of the Ground Floor Elements to Prevent Weak Story Collapses

A weak or soft ground story causes the deformations due to seismic loading to concentrate in the ground story, while often the upper stories have small inter-story drifts such that the portion of the building above the ground story behaves as a rigid body[7].

The weak-story mode dominates the dynamic response of the building, and the resulting damage concentration in the weak story often leads to collapse. As the first story is strengthened, the upper stories experience higher forces and drifts. Therefore, making the first story too strong relative to the upper stories can cause excessive drifts and weak-story collapse of the story above. Even for new construction, it may not be economical to employ extra strengthening measures in the next story above

[7] See Section 1.2 for a more complete discussion about weak and / or soft stories.

ground. Considering this, strength of the ground floor elements must be "tuned" to the strength of the story above, in order to obtain a design that prevents a weak-story collapse in either story.

Research conducted to develop guidelines for certain weak-story, wood frame buildings in California, USA (FEMA P-807, 2012) found that there is a "sweet spot" of optimized ground-story element strengths, as Figure 36 shows. The different lines in Figure 36 correspond to analyses of the same structure, but with different strength levels for the first story. (Somewhat surprisingly, stiffness was found to be less important than strength in determining the response of these buildings.) Strengthening the first story reduces the drifts in the first story, but increases the drifts in higher floors. As shown, a point exists beyond which adding more strength in the ground story simply shifts the failure location to the story above, and global behavior does not improve. The key is to provide enough ground-story strength to prevent a weak-story collapse at ground level, but not enough to cause unacceptable levels of damage in the stories above. It is also important to check in-plane, out-of-plane interaction for panels with large height-to-thickness ratios. (See the In-Plane, Out-of-Plane Interaction section in Strategy A.)

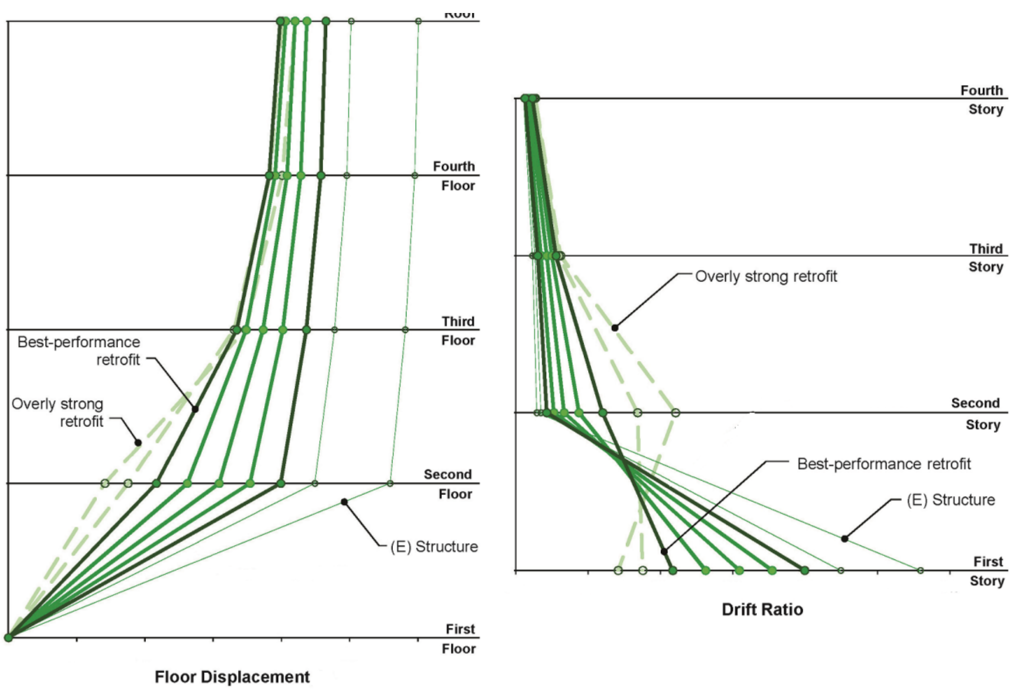


Figure 37. Plots illustrating floor displacement and drift ratios in a building with constant upper-story strength and varying first-story strength. (E) Structure = Existing structure. (Reproduced from FEMA P-807)

For the tuned-strength approach to work properly, the designer must provide a ground-story strength that is between the minimum and maximum ground-story strength values that will give acceptable behavior. Per FEMA P-807 (a freely available document), the minimum ground-story strength is the strength required to reduce the ground-story drifts until they satisfy the maximum ground-story drift limit, which is set a point intended to safeguard against collapse. The maximum ground story strength is the strength that, if exceeded, would cause the upper stories to exceed their acceptable drift limits. The ranges of acceptable first-story strengths should be determined separately for each principal direction. Though FEMA P-807 was developed to provide guidance for retrofits in timber buildings, the concepts are general and also apply to new buildings and other materials. Caution should be used in applying these concepts to taller buildings; FEMA P-807 studied buildings up to four stories tall. The following practical rule of thumb from FEMA P-807, in which n is the number of stories, can be used as an upper bound on the ground story strength to prevent the ground story from becoming too strong and causing a weak-story mechanism to form in the story above:

$$\frac{ground\ story\ strength}{2nd\ floor\ strength} > \frac{n}{n-1} \quad (1c)$$

For example, this ratio is 1.33 for a four-story building. This practical rule was derived from general strength balancing principles discussed in Strategy A and validated by extensive nonlinear time history analyses. The lower bound on the ground story strength, that is, the strength needed to prevent an unacceptable level of deformation in the ground story, must be determined by structural analysis.

Options for Adding Strength and Energy Dissipation

The most straightforward way to add strength to an open story is to provide a properly detailed reinforced concrete shear wall of the requisite length and thickness. As discussed above, the wall strength must be tuned to prevent failure in the adjacent story. In cases where a reinforced concrete shear wall would be too strong, and cause failure to occur in the story above, specially-designed panels can provide the required amount of strength and dissipate energy.

Figure 37 shows a special type of “grill” panel that can be used to strengthen the ground story and to provide a means of controlling this strength. This type of panel was developed to allow tuning of the ground floor strength, in a case where a reinforced concrete shear wall would have been too strong, forcing failure in the story above. As illustrated in Figure 37, reinforced concrete parts between the vertical slits in the wall act as columns under lateral deformations. Appropriate reinforcement detailing in these columns enables the formation of plastic hinges, which dissipate energy and control the shear strength of the wall. Despite the detailing required for implementing this option, it can be significantly less expensive than other alternatives, such as buckling restrained braces (BRBs).

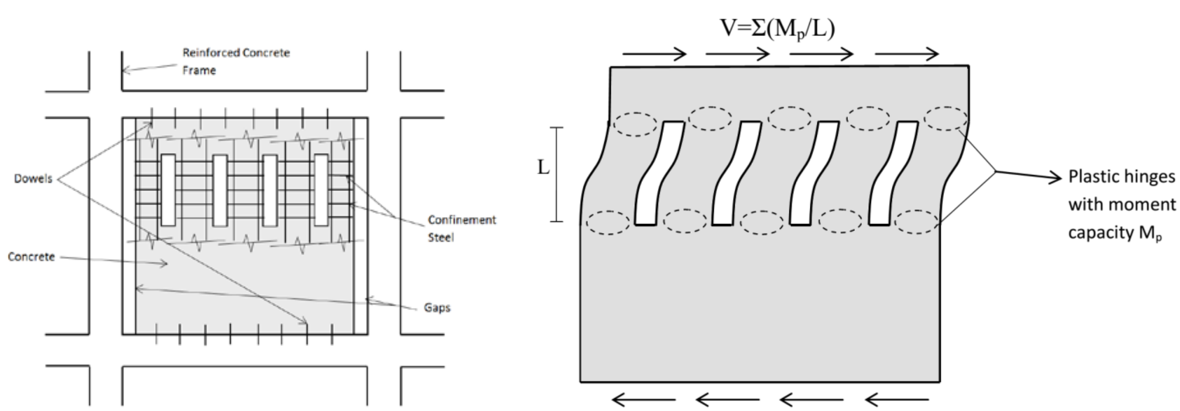


Figure 38. Option for a wall element adding strength to the ground story (left); deformed shape of the wall demonstrating the formation of plastic hinges and maximum shear strength of the wall (right). Based on a design by Tipping + Mar.

Another low-cost option is a damped infill wall panel, which is an augmented infill panel that dissipates energy through damage or sliding. The simplest would be a type of friction damper, where intentionally weak mortar promotes bed-joint sliding between masonry units. As illustrated schematically in Figure 38, energy dissipates through friction generated as the masonry units slide. The designer has some controls over the shear strength of the damped infill wall panel by specifying the mortar strength. The panel must be held together out-of-plane, to prevent masonry units from falling out. Figure 38 shows a simple detail for restraining elements out-of-plane, with geotextile fabric or other mesh material on both wall faces and connected with crossties. Fabric and mesh have been used in adobe and other masonry buildings (e.g., Neumann et al, 2007).

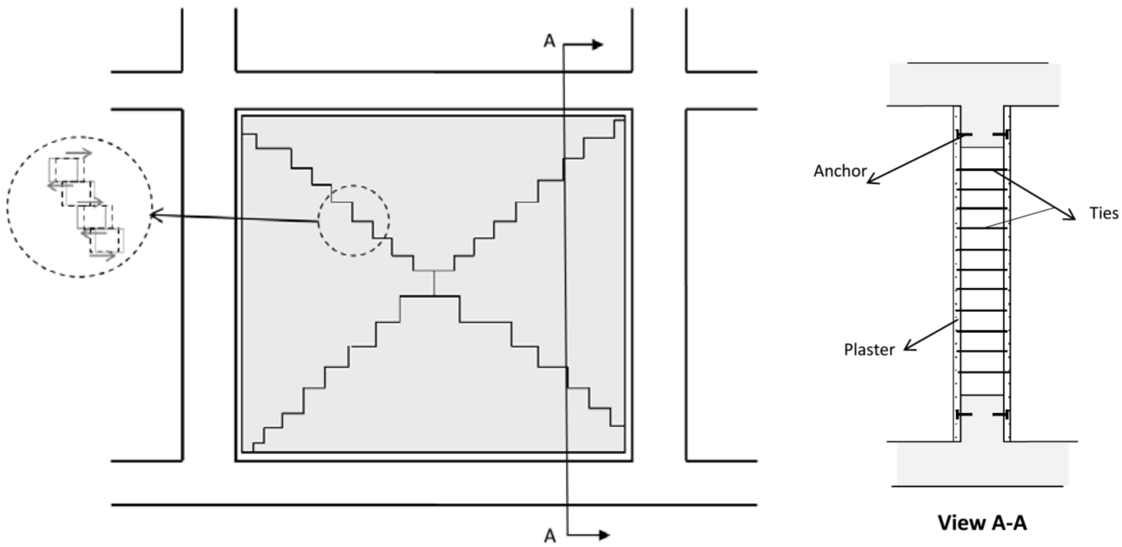


Figure 39. An option for controlling the shear strength of the wall using frictional force between the small pieces (left); detail for providing out-of-plane stability (right) failure

Options for Adding Energy Dissipation

In higher income countries where infill buildings are common, adding passive energy dissipation devices – often informally referred to as "dampers" – in the ground story may be a potential alternative. Energy dissipation devices are more commonly used in the seismic retrofit of buildings, but have been used in new construction, primarily for steel frame buildings. Passive energy dissipation devices increase damping (and sometimes stiffness of the building) and reduce the energy dissipation demands on the structural members. Larger reductions can be achieved if these devices add to the stiffness of the building. However, when only limited performance objectives, such as collapse prevention, are desired, energy dissipation systems may not be appropriate design strategies. These systems are generally more applicable to retrofit of buildings when superior earthquake performance is desired and associated costs can be afforded. In this case, the costs are offset by eliminating the need for using strengthening and stiffening methodologies to meet the performance objectives.

A number of different energy dissipation devices have been developed for seismic protection, including fluid dampers, viscoelastic solid dampers, friction dampers, metallic dampers, Buckling Restrained Brace (BRB) dampers, and added damping and stiffness (ADAS). Further information and details regarding design uses can be found in Symans et al. (2008) and Izumi et al. (2004).

Practical Guidance to Determine Whether a Ground-story-only Solution is Appropriate

This design strategy is intended for buildings with relatively few infill panels in the ground story compared to the upper stories. Residential buildings with an open ground story for shops or parking are especially good candidates. This strategy works best in cases where, if nothing were done, the ground story would be significantly weaker than the upper stories.

The approach described in this section is conceptually applicable to shear, side-sway buildings, which include the reinforced concrete moment frames targeted in this document. (This approach is not applicable to buildings with a global flexural mechanism, such as tall, shear wall buildings.) However, in order for this approach to work, the prevailing mode of structural failure must be formation of a weak-story mechanism, and the building must not be prone to significant overturning and axial failures in columns, especially in taller buildings. The upper stories must not be prone to significant torsion; otherwise, strengthening the ground story may increase displacements, and thus worsen the effects of torsion in upper stories.

As explained in the previous section, the minimum required strength of the ground story is that needed to satisfy the specified performance objective in terms of the ground-story drifts, and the maximum acceptable strength of the ground story is the strength at which upper-story drift exceeds the acceptable objective. Currently, this strength is best determined by a nonlinear static analysis, but future efforts similar to FEMA P-807 could provide simplified methods for designers. If these two bounds happen to be contradictory due to overall weakness of the structure, strengthening merely the ground story will not satisfy the performance objectives and will lead to exceeding the maximum drift limits required to satisfy the desired performance level.

If the upper stories are so weak that strengthening the first story results in large drifts in the upper floors, another design strategy, such as a rocking spine (Strategy B), should be selected. Otherwise, as an alternative, an owner or jurisdiction might consider an option of providing optimized first-story strength within 10% below or above the maximum bound, as suggested by FEMA P-807, with provisions made to reduce torsion.

Layout of Walls to Reduce Torsion

When tuning ground story strength, the designer must pay attention to torsion in the ground story itself, as well as within the upper stories. The latter involves a straightforward check, while the former requires more design attention. In general, walls subjected to torsion undergo significant drift ratios that will dramatically decrease their in-plane lateral strength, thus leading to collapse under smaller earthquake loads. Furthermore, story torsion will amplify the out-of-plane effects because of increased accelerations due to torsion. The walls will become damaged and degrade more quickly due to the additional displacement demands torsion creates. Accordingly, significantly higher capacity can be achieved with the same amount of material when lateral-force-resisting elements are located in plan so as to minimize torsion. FEMA P-807 (section E.6.5) provides detailed information about the effect of torsion on response capacity.

In order for ground-story-only interventions to be effective, upper stories must not be prone to significant torsion, as explained in the previous section. This design strategy assumes that the upper stories behave as a near-rigid box compared to the ground story, both in translation and torsion. To prevent upper stories from being subject to significant torsion, inertial forces from upper stories need to be transferred to the first story near the geometric center of the second floor (FEMA P-807). Figure 39 illustrates an approximate rule of thumb suggested by FEMA P-807. This condition may be considered satisfied if, in each upper story, the distance from each story's center of strength to the center of mass of the floor below is no more than 25% of the corresponding building dimension. (Center of strength is defined as the location of the force resultant of the wall strengths, with more details in Section 4.6.4 of FEMA P-807.)

Achieving the optimized, "tuned" story strength is more important than achieving the minimum eccentricity. Even if the eccentricity can be eliminated by adding more or stronger elements, it should not increase the strength of the first story beyond the maximum limits as the previous section describes.

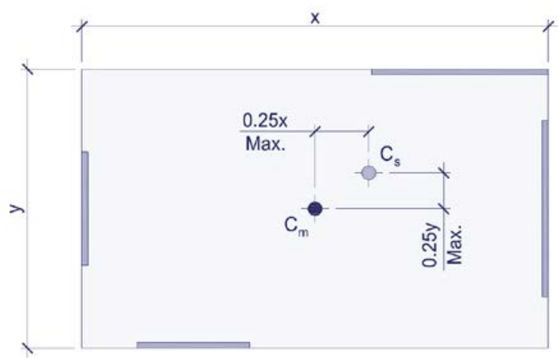


Figure 40. Floor plan schematic showing limits on eccentricity between the center of strength (C_s) of each upper story and the center of mass (C_m)of the floor below it. (reprinted from FEMA P-807)

The designer can address torsion in the ground story itself through two possible approaches. The first, and most straightforward, is to lay out the ground story's lateral-force-resisting elements in a way that minimizes torsion. The second approach, which might be necessary due to architectural or functional constraints, is to design ground-story, lateral-force-resisting elements for significant additional toughness, so that they can accommodate the extra cyclic deformation demands created by the torsional response. Figure 40 shows three buildings with different weak-story designs and, therefore, different levels of sensitivity to torsion. Figure 41 and Figure 42 illustrate the cases where earthquake loading in only one direction can create torsion, and where earthquake loading in both orthogonal directions creates torsion, respectively. As shown, corner buildings that have a line of infill walls on the two property line boundaries will require significant design efforts to reduce torsion to levels that the ground-story lateral elements can accommodate safely.

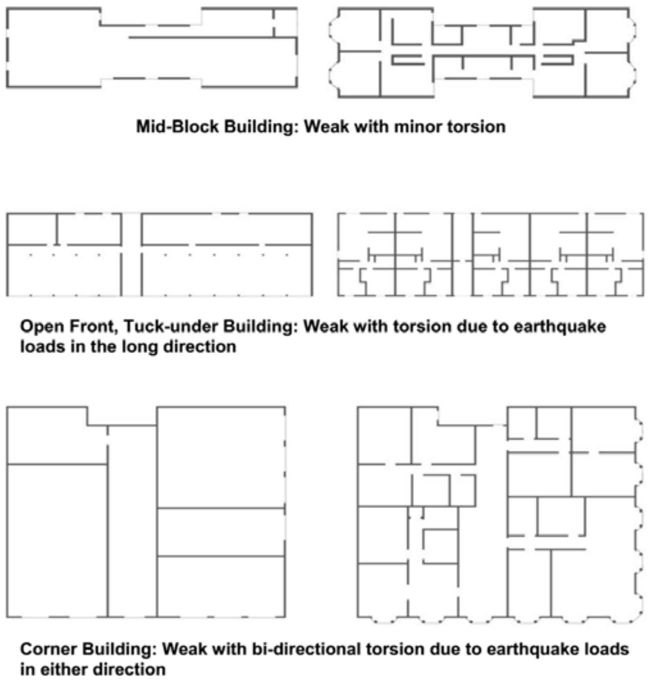


Figure 41. Plan diagrams of three weak-story buildings. Left: first story; Right: upper stories. (reprinted from FEMA P-807)

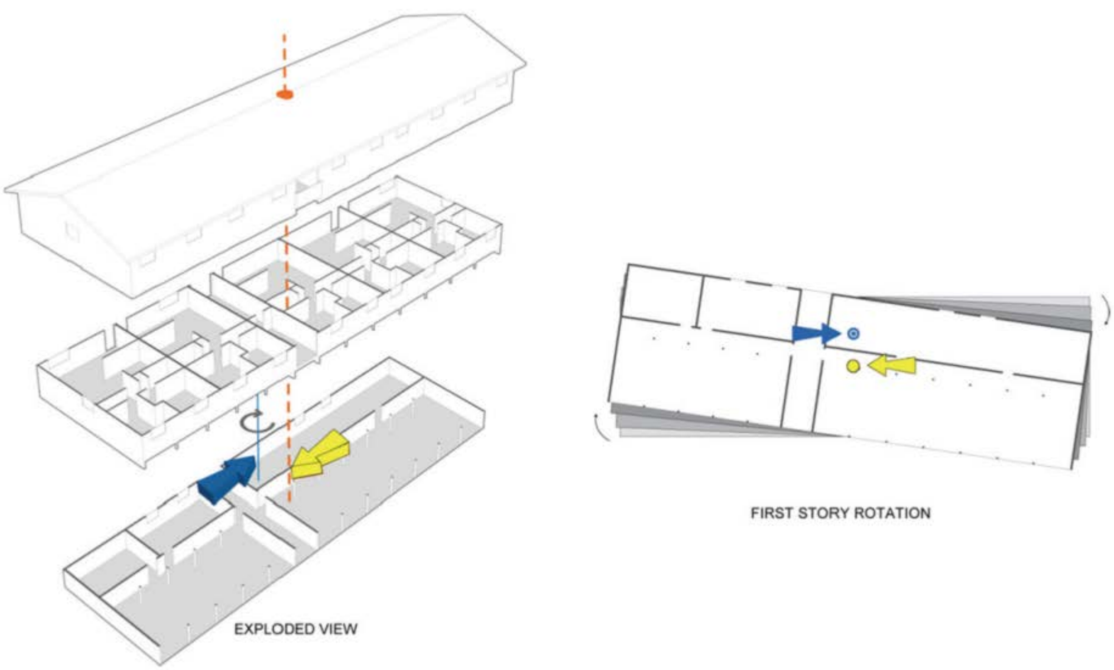


Figure 42. Schematic response of a building with a slightly weak story in the long direction and with torsion under loads in the long direction. The distance between the arrows indicates the eccentricity that gives rise to torsion in the first story. The light arrow denotes the center of strength of upper stories; the dark arrow represents the center of strength of the first story. (Reprinted from FEMA P-807)

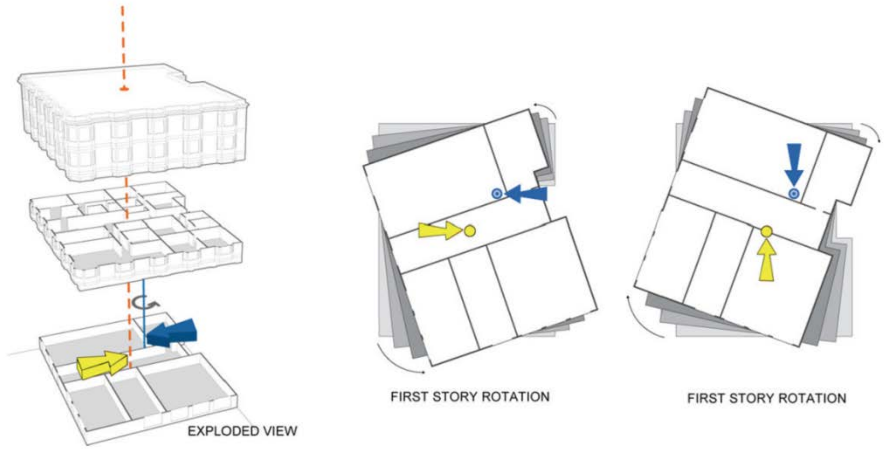


Figure 43. Schematic response of a corner building subject to torsion under loads in either direction, due to open wall lines on two sides at the first story. The distance between the arrows indicates the eccentricity that gives rise to the torsion in the first story. The light arrow denotes the center of strength of upper stories; the dark arrow represents the center of strength of the first story. (Reprinted from FEMA P-807)

Detailing and Construction Considerations

Once the designer determines required strength of elements added to the ground story using the concepts explained in the previous sections, members should be sized based on expected material strengths (including overstrength), and on the full expected capacity of the member, without the strength reduction factors typically used in new design. Because making the ground story elements too strong will damage and possibly collapse the second story, it is critically important to fully account for ground story member strength.

Detailing proceeds as in typical design for special seismic resisting elements, using the appropriate standards and/or guidelines and using capacity design concepts to ensure a complete load path that confines damage to ductile behavior in the ground story. Requirements and considerations for detailing reinforced concrete, reinforced masonry, and masonry/reinforced concrete composite walls can be found in the following standards and guidelines:

- Reinforced concrete shear wall: National and international reinforced concrete standards including American Concrete Institute Standard 318, Eurocode 8, Indian Standard 13920, and many other national codes; guidance on expected properties can be found in ASCE 41/FEMA 356.
- Reinforced masonry wall: Building code requirements and specifications for masonry structures (TMS 402-11/ACI 530-11/ASCE 5-11), Eurocode 8, Mexico's NTC-M 2002, IITK-GSDMA (2005), ASCE 41/FEMA 356.
- Masonry/reinforced concrete composite walls: TEK 16-3B

Concrete infill walls may also be used, but standards are not readily available. Guidelines for including energy dissipating devices in new buildings can be found in national standards in countries where energy dissipation is used extensively, such as Japan and New Zealand.

Modeling Reinforced Infill with Equivalent Tension and Compression Struts

Reinforced masonry infill walls, or infill panels strengthened with mesh reinforcement and shotcrete / gunite can be simply modelled with two diagonal struts rather than one. In nonlinear static analysis, one member will always be in compression while the other will always be in tension. Figure 43 shows a model with both tension and compression struts.

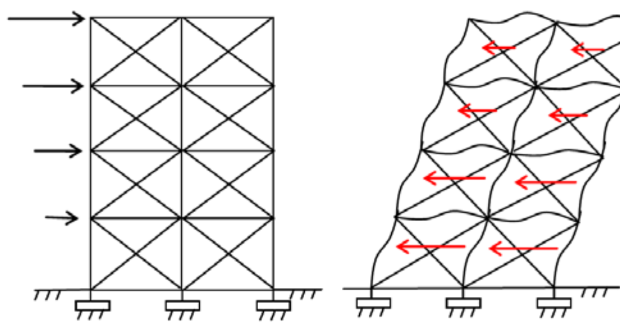


Figure 44. Model with tension and compression struts for reinforced infill panels

The equations below are used for calculating the effective width *a* and axial stiffness k_{inf} of an equivalent strut, which is then distributed equally to the two struts.

$$\lambda_1 = \left[\frac{(E_m \cdot t_{\inf} + E_c \cdot t_c) \cdot \sin(2\theta)}{4 \cdot E_{fe} \cdot I_{col} \cdot h_{\inf}} \right]^{0.25} \tag{2c}$$

$$a = 0.175 \cdot (\lambda_1 \cdot h_{col})^{-0.4} \cdot L_{diag} \tag{3c}$$

$$k_{\inf} = \frac{a \cdot (E_m \cdot t_{\inf} + E_c \cdot t_c)}{L_{diag}} \tag{4c}$$

E_m, elasticity modulus of the infill

E_c elasticity modulus of concrete used for retrofit

E_{fe} elasticity modulus of the frame material

t_f thickness of the infill wall

t_c thickness of the concrete

θ angle of the diagonal strut with the horizontal

h_{col} height of the surrounding frame column

I_{col} moment of inertia of the surrounding frame column section

h_{inf} height of the infill wall panel

L_{diag} length of the diagonal strut

The strengths of the compression and tension equivalent struts are given by:

$$Ncomp = \frac{A_{\inf} \cdot fs_{\inf} + A_c \cdot 3.3 \cdot \sqrt{f_c}}{\cos\theta} \tag{5c}$$

$$Ntens = \frac{A_s \cdot f_{ys} \cdot L_{\inf} / s}{\cos\theta} \tag{6c}$$

A_{inf} cross sectional area of the infill wall (in^2)

A_c cross sectional area of concrete (in^2)

fs_{inf} shear strength of masonry

f_c concrete strength (psi)

θ angle of the diagonal strut with the horizontal

A_s total cross sectional area of horizontal mesh reinforcement with spacing s

f_{ys} strength of steel

L_{inf} wall length

Other elements added to the ground story as a means of adding strength can also be modelled as struts, by using the backbone curve derived from test data in order to define the strut properties.

5.3 Capacity Design Checks

Capacity design principles must be applied to ensure that an appropriate load-path exists for transferring loads from the second story (first story above ground) to the ground-story lateral load resisting elements, and to their foundations and the supporting soil. Elements in the load path, such as connections to the second-story diaphragm, must be designed to develop sufficient strength to prevent failure modes other than the intended ductile failure mechanism from occurring. Therefore, load path elements other than the main lateral-force-resisting element (i.e., the shear wall or damped infill panel) should be designed with appropriate strength reduction factors, though such factors cannot be used for the main lateral-force-resisting element(s). Foundations should be designed to resist bearing, sliding and overturning that will result when the main lateral-force-resisting elements act at full strength.

Ground story columns must be designed for deformation compatibility and detailed to provide a high level of ductility to prevent nonductile failure modes, such as shear failure and axial crushing. Columns should have the ductile detailing prescribed for special seismic moment resisting frames in reinforced concrete design standards, which includes confinement reinforcing of plastic hinge zones, splices and in beam-column joints. Horizontal components of the forces acting on the struts that are used to model ground story elements do not need to be considered for the shear design of columns, since the separation of the element or infill from the column is prevented and the horizontal forces are transferred to the frame not only from two points but all along the perimeter of the panel.

6 Design Strategy D: Separation of Infill Walls from Frame

6.1 Motivation and Design Rationale

Separating the infill walls from the frame, by leaving a seismic gap on the sides and top to accommodate the expected frame deformation, allows designers to rationally and accurately use the common assumption that the frame is a bare frame for analysis and design. This approach also preserves architectural flexibility and structural safety in situations where infill panel locations may change significantly over the life of the building, such as in office buildings.

Also, for cases where significant torsional irregularities caused by infill walls are costly or impractical to eliminate, such as in corner buildings, isolating the infill panels from the surrounding frame may be a better alternative. However, there are significant challenges involved in properly isolating infill walls, many of which are related to constructability, weatherproofing and fire resistance.

6.2 Concepts and Implementation Strategies

Requirements for Proper Isolation of Infill Panels In-plane

Infill panels considered isolated from the surrounding frame must have gaps at top and sides to accommodate maximum expected lateral frame deflections (Section 7.5.1, FEMA 356). NZS 4230 requires the gaps to accommodate ultimate limit-state, inter-story deflections. As a guidance, TMS 402-11 requires that where the specified gap between the bounding beam or slab at the top of the infill is less than 9.5 mm (3/8 in.), or the gap is not sized to accommodate design displacements, the infill shall be designed as a participating infill (i.e., a structural element) in accordance with Sections B.3.4 and B.3.5, except that the calculated stiffness and strength shall be multiplied by a factor of 0.5.

Gaps must be free of materials that could transfer loads between the infill and bounding frame, such as mortar, debris and other rigid materials, even if these materials are weak (TMS 402-11). The material used to fill these gaps must be capable of accommodating frame displacements, including inelastic deformation during seismic events, provided that the compressibility of that material is taken into account for calculating the required size of the gap. Highly flexible materials, such as polystyrene (Paulay and Priestley, 1992), can be used. When isolating infill panels, special details may be required to meet weather proofing, insulation, fireproofing or soundproofing requirements.

Concerning isolation of the infill panels from the surrounding frames, New Zealand Standard for Design of Reinforced Concrete Masonry Structures (NZS 4230, 2004) warns designers that "even where sufficient separation is provided at top and ends of a panel, the panel will still tend to stiffen the supporting beam considerably, concentrating frame potential plastic hinge regions in short hinge lengths at each end, or forcing migration of hinges into columns, with a breakdown of the weak-beam, strong-column concept. When infill panels are constructed without full separation from the frame, the composite action must be considered in analysis and design accordingly. Structural stiffness is greatly increased, and natural periods reduced. This is significant when determining the appropriate basic seismic coefficient."

Infill walls appropriately isolated from the frame are referred to as non-structural components, non-participating infills, or isolated infills in various standards. Isolated infill walls need to be designed for inertial forces caused by seismic acceleration, and prevented from failing out-of-plane. The next section provides guidance on how to calculate these forces.

Preventing Out-of-plane Failure

With a seismic gap, the panel is no longer supported out-of-plane by the frame itself, and must be anchored to prevent toppling during strong shaking. Since the arching action is eliminated when the panel is isolated from the frame due to the flexibility of the isolating material, isolated panels must be reinforced to carry the out-of-plane forces (Paulay and Priestley, 1992). Out-of-plane forces on infill panels can be calculated using the equations that are typically found in the "nonstructural components" or "nonstructural elements" provisions in various standards. Designers should use the standards in force in their country of practice, but if that country's standards do not contain nonstructural provisions, then designers should use standards from elsewhere that contain such provisions. Here is one equation for calculating out-of-plane forces, from Eurocode 8; other standards contain similar provisions:

$$F_a = S_a W_a \gamma_a / q_a \qquad \text{where seismic coefficient } S_a = \alpha S \left[\frac{3(1+z/H)}{1+(1-\frac{T_a}{T_1})^2} - 0.5 \right] \qquad (1d)$$

And where

F_a	horizontal seismic force, acting at the center of mass of the panel
W_a	weight of the panel
γ_a	importance factor of the panel, equal to 1.0
q_a	behavior factor of the panel, equal to 2.0
α	ratio of the design ground acceleration on rock (Type A) to the acceleration of gravity g
S	soil factor
T_a	fundamental vibration period of the component
T_1	fundamental building period in the relevant direction
z	height of the panel above the top of foundation or top of rigid basement
H	building height measured from top of foundation or top of rigid basement

There are multiple details to resist the above forces and prevent the panel from toppling out-of-plane. The sections below describe several.

Divide Larger Panels into Smaller Ones, and Reinforce the Infill Panel at Regular Spacing in the Vertical and Horizontal Direction

Providing stiff members made of wood or lightly reinforced concrete in vertical, diagonal and/or horizontal directions divides large masonry infill wall panels into smaller parts and increases wall stability (Murty et al., 2006).

Some countries (e.g., Indonesia and Mexico) have design codes to accomplish this purpose without interfering with the frame members. These codes recommend providing lightly reinforced concrete columns, also called *practical columns*, with a small cross-section and vertical steel bars loosely inserted into the beam at the top end, at regular intervals along the wall length and at the wall ends, as illustrated in Figure 44 (Murty et al., 2006). Practical columns and beams are typically difficult to construct because the small member sizes make it challenging to place the reinforcing steel and concrete correctly.

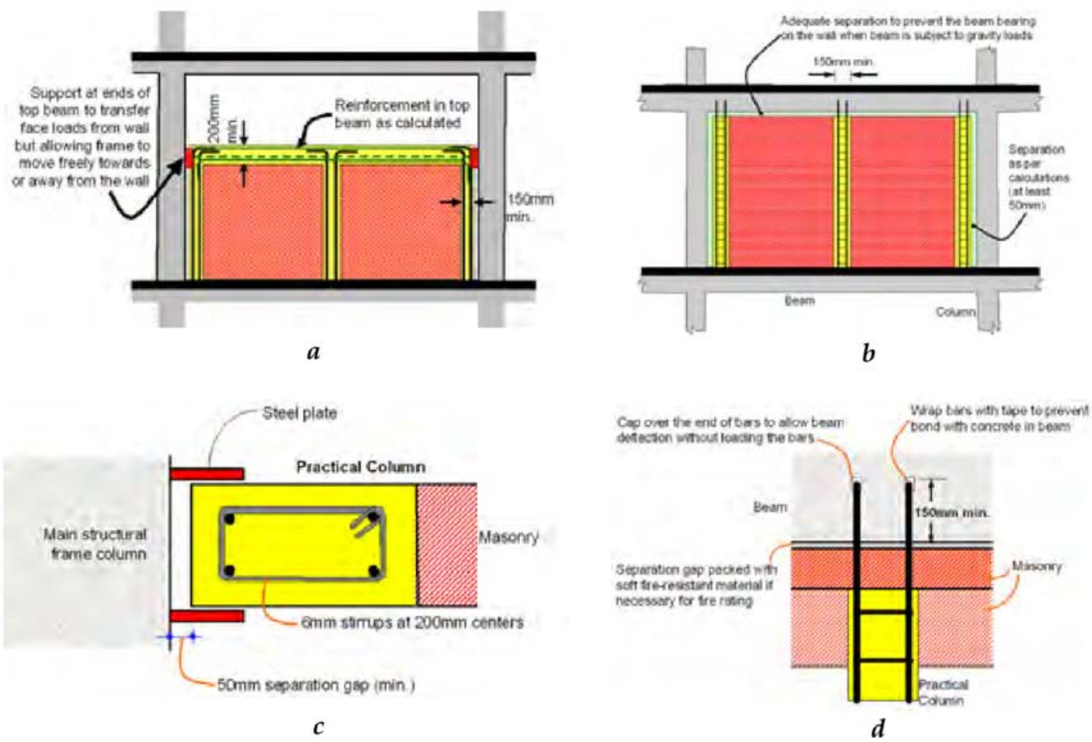


Figure 45. Practical columns provided to isolate masonry infill walls in Indonesian practice, (a) partial height infill, (b) full height infill, (c) close-up details of a practical column, and (d) close-up details of anchoring practical columns into the beam above. (Source: Murty et al., 2006)

Design Shear Connections

Provide shear connections between frame and panel by extending panel vertical reinforcements into the beam and taping layers of flexible material (e.g., polystyrene) to the sides of the reinforcement in the in-plane direction, up to the beam mid-height. This will restrict out-of-plane relative movements while allowing in-plane movements (Paulay and Priestley, 1992).

Install a Supporting Column

Columns attached to the walls can act like splints or strongbacks that brace the wall against excessively bowing outward or inward, as illustrated in Figure 45 (FEMA P-774). This solution can also be used to brace partitions not inside the frame.

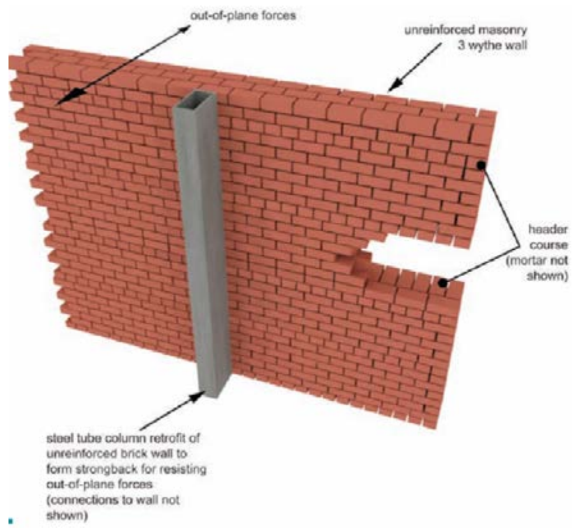


Figure 46. Schematic representation of a strongback column spanning from foundation to roof, serving to brace a brick wall against out-of-plane forces. (Consortium of Universities for Research in Earthquake Engineering, Figure reprinted from FEMA P-774)

Provide Steel Top Angles or Similar Mechanical Connectors

Out-of-plane resistance with an in-plane slip joint can be provided by attaching continuous or intermittent steel angles to the beam or slab above, as shown in Figure 46. Steel plates or other similar mechanical connectors could be used in similar fashion. Detailing must be done carefully for interconnected perpendicular walls, so that the out-of-plane restraint for a wall will not create an in-plane restraint for a wall perpendicular to it. The recommended spacing for intermittent angles or other connectors is no more than 1200mm (48 in) on center (TMS-402). In unreinforced masonry infill walls, angles or connectors should extend downward to more than one masonry unit to help prevent the situation observed in earthquakes in Mexico, where the angles only retained a portion of the wall, such as the top brick, and did not prevent out-of-plane failure.

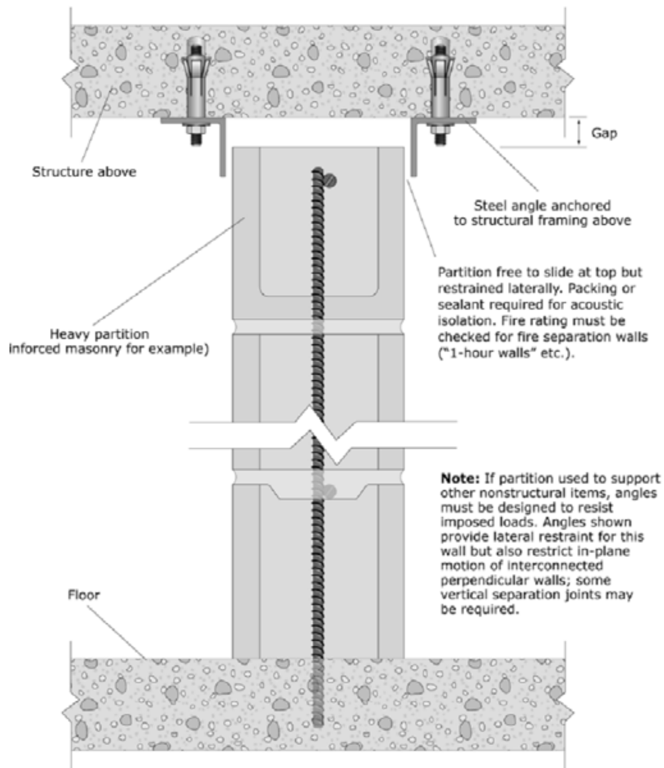


Figure 47. Details for providing out-of-plane resistance along isolated, reinforced infill using steel angles (Reprinted from FEMA E-74)

Considerations for Weather, Fire Resistance, Durability, Architectural Finishes

Leaving a seismic gap between the infill panel and frame creates a number of issues for fire protection and weather resistance. The gap must be filled with a material that effectively seals the gap, preventing moisture intrusion and excessively hot or cold air (for climate controlled buildings). This material must also resist fire and provide the appropriate fire rating. Small earthquakes can crack and damage the material filling the gap, lessening the fire resistance and allowing water intrusion. Building owners may also be displeased by such damage. If architectural finishes such as plaster cover the gap, they may allow some force transfer and defeat the panel's isolation. In addition, some porous materials can absorb a water cement mix from the mortar, which significantly reduces or eliminates its flexibility, and the wall is no longer isolated from the frame.

7 Design Strategy E: Change the Structural System

In some cases, a frame with masonry infill will not be the best choice of structural system, and it may be feasible, from an economic, regulatory and constructability standpoint, to choose a different structural system. As discussed in the introduction, the current state of design and construction practice in a number of countries tends to make frames with infill the preferred structural system for most buildings, and certainly for taller ones. However, even in these cases it can be possible to change the structural system if a frame with infill will not be appropriate.

7.1 Alternate Systems

Two common structural systems that can be used in place of infill buildings are:

- Confined masonry
- Reinforced concrete shear wall

Depending on local construction practices, reinforced masonry may also be a viable alternative system. Alternate systems must be properly designed to overcome any configuration irregularities created by the building's architectural and functional design. Because the alternate systems are shear wall systems, a proper wall layout that reduces torsion and avoids severe vertical irregularities such as discontinuous shear walls is crucial for achieving acceptable seismic behavior.

A number of countries have building codes or standards for confined masonry. The Confined Masonry Network website (www.confinedmasonry.org) provides a list of codes for confined masonry, as well as design and construction guidelines (i.e., Meli et al., 2011) and other reference materials on confined masonry. The reinforced concrete code in many earthquake-threatened countries also contains ductile seismic detailing requirements for shear walls. The World Housing Encyclopedia's tutorial on concrete frames with masonry infill provides additional guidance on selecting an alternate system.

7.2 When to Consider an Alternate System

The major consideration in determining whether to change the structural system is whether the alternate system will provide a more functional and economical design solution, taking into account the local construction, regulatory and economic environment. The quality of masonry construction versus concrete construction is also a consideration; in locations with very poor concrete construction practices but good masonry construction practices, confined masonry is more likely to be built properly. For single story and two story (i.e., ground plus one story) buildings, confined masonry is likely to be more economical than infill frame construction. This is especially likely to be true in countries where local builders are familiar with confined masonry construction.

For buildings taller than six (ground plus five) stories, the bearing wall nature of the structural system makes the use of confined masonry or reinforced masonry impractical, so the only viable alternative system would be a reinforced concrete shear wall. The Confined Masonry Network is developing guidelines for engineered confined masonry that, when completed, can be used to design confined masonry buildings up to six stories tall. Shear walls can be used for a wide range of building heights but are more likely to provide the most economical solution for medium-height to high-rise buildings (Murty et al., 2006), especially in areas of high seismic hazard. In areas where concrete frame construction quality is very poor, and design Strategies A, B and C in this document are not practical or feasible, a reinforced concrete shear wall system can be a good alternative.

8 Concluding Remarks

Concrete frames with masonry infill walls are one of the world's most common building types. The design strategies contained in this document are intended to reduce, in these buildings, the common seismic vulnerabilities that have caused damage in recent earthquakes around the world. These are by no means the only possible strategies to improve the earthquake performance of frames with infill. The writers of this document intend that it be a living document. They encourage the international earthquake engineering community to discuss and improve upon the strategies presented, and welcome colleagues around the world to develop and submit additional strategies.

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