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9 Stability, robustness and movement joints

The IStructE *Practical guide to structural robustness and disproportionate collapse in buildings*[92] states that:

> *"robustness is not a commodity readily defined".*

However, detailed reading of the guide shows that robustness can loosely be split into three categories:

- Lateral stability (as opposed to local stability and buckling)
- Robust design and details
- Disproportionate collapse

While it is not possible to go through each of these in detail at conceptual design stage, many of the decisions made will impact them. As a result, a clear and well-considered approach to each early on will save engineers a lot of time later in the design process.

This chapter provides guidance for low- and medium-rise buildings. The stability design of high-rise buildings is a specialist area beyond the scope of this book.

9.1 Lateral stability

While it could be argued that the provision of lateral stability is not so much a case of robust design, but of design more generally, as in the design of beams and columns, we would argue that the function of lateral stability *is* to ensure a robust design. In simple terms, lateral stability prevents the collapse of buildings subjected to lateral loads. If inadequate lateral stability is provided, the effect is not just the failure of the bracing or shear wall, as it would be for a beam or column, but the failure of the entire building. Lateral stability, while a minor part of the cost and material use of the building, is a major part of the structural design of the entire building.

As with all other elements of design, the aim at the concept stage of the project is not to complete a full design of the building's lateral stability, but to do enough to convince yourself, as an engineer, that the solution is feasible and should work. Depending on the type of lateral stability provided, this might be as simple as looking at a plan, or it may require quite significant calculation, even at the early stages. This chapter will help highlight:

- different stability mechanisms
- what is an appropriate level of design at this stage for the proposed solution
- how to approximately calculate the associated loads

Ultimately, the aim is to be able to conceptualise in three dimensions how the lateral stability works, and communicate this to the rest of the design team.

9.1.1 Horizontal loads

Horizontal loads can be separated typically into:

- internally applied loads
- externally applied loads

Internal loads are caused by a variety of factors e.g. out-of-plumb of the building and lack of fit. These forces are typically dealt with as lateral loads, known as ‘equivalent horizontal forces’ (EHF) in the Eurocodes, or ‘notional horizontal loads’ (NHL) in British Standards, rather than through the load eccentricity of element loads, although other approaches are possible.

Externally applied loads include earthquakes, explosions, large horizontal impacts and wind loads. In the UK, only the last of these typically needs to be considered at concept design stage, however accidental loads in the form of impacts and explosions may need to be considered when thinking about the effects of disproportionate collapse. For buildings in seismic regions, the consideration of earthquakes is as important, if not more important, than wind loading, depending on the region. The consideration of loads due to earthquakes is beyond the scope of this book but *Seismic design of buildings to Eurocode 8*[93] and the IStructE *Manual for the seismic design of steel and concrete buildings to Eurocode 8*[94] provide a comprehensive description of the design process.

Typically, external loads result in a reaction at foundation level, equal and opposite to the applied external load. However, internal loads, if a function of gravity loads only, must be resolved within the building (sometimes using the foundations), and do not exert horizontal loads on the ground around.

Estimating wind loads

There are three levels of wind load estimate (note these are UK-specific but a similar approach may be suitable elsewhere):

- **The five second guess**
  This is useful when you need a really quick answer. It won’t be correct, but it will be a good guess. Most laterally applied loads are in the order of 1kN/m^2. To give some conservatism, you could use 1.3kN/m^2, and if your building is on high ground or near the coast (especially if exposed to south-westerly winds), you could use 1.5kN/m^2.
- **The ten minute calculation**
  If you have a little longer, it isn’t very hard to get a conservative wind load. You need to take the following steps:

  Step 1 using Figure NA.1 from the National Annex to BS EN 1991-1-4[95], locate your building and choose $v_{b,map}$
  Step 2 using NA.2.5, calculate c_{alt} using equation NA.2a (for buildings taller than 10m the other equation gives a less conservative value). Unless your building is temporary or going to be standing for a very long time you can ignore c_{season} and c_{prob}. This equates to c_{season} and c_{prob} both being equal to 1.0. Likewise, we will ignore wind direction as it almost always reduces the wind speed. We can now multiply $v_{b,map}$ by c_{alt} to get our final, conservative wind speed
  Step 3 calculate the pressure using $q_b = 0.625.v_b^2$ (Equation 4.10 of BS EN 1991-1-4[96])
  Step 4 choose the worst distance to town and sea and use these values to look up the $c_e(z)$ and $c_{e,T}$ from Figure NA.7 and NA.8 in the National Annex (if you can’t work out the distance to the edge of town, you can assume it is zero as this will be conservative)
  Step 5 multiply these values and the basic wind pressure to get the site wind pressure
  Step 6 using Table NA.4, calculate the building c_p. If you don’t have time to calculate *h/d* or it is not clear use 1.3. This gives the lateral load on a roughly rectangular building in kN/m^2

At this stage, it’s worth noting that if your building is an open canopy, or wall, dome, cylinder or giant asymmetric blob, there are other pressure coefficients you should use. Some are given in BS EN 1991-1-4, but often reference to Cook[97] will be required. For most structures, the horizontal load will be less, as the shape is more aerodynamic than a rectangular cuboid. The notable exception is a wall which has particular properties under wind load. For uplift, structures such as open canopies exhibit larger values than bluff bodies such as rectangular buildings, but this won’t affect the lateral load.

- **The one hour calculation**
  If you have time, you can do a full wind calculation. It doesn't take long and you will need to do it at some point. Most offices either have access to an automated calculation package, or will have their own spreadsheet that can do this, and produce pressure maps for the 12 directions of wind load. Once the building location is agreed to a set site, the site values will not change even if the building moves or changes orientation.

Estimating equivalent horizontal force

BS EN 1991-1-4 provides detailed guidance for EHF, depending on the number of floors and columns. However, as a simple guide 1/200 of the vertical load should be applied as an EHF. For many buildings, EHF is a small proportion of the total applied horizontal load, and can be ignored at the concept design stage. However, this is not always the case. In the following circumstances, EHF should be considered at the concept design stage:

- If there is no wind load e.g. for mezzanines or new structures infilling an existing courtyard
- If the building is tall and thin e.g. the Flatiron Building in New York only attracts a small amount of wind load on its narrow face but the EHF remains the same regardless of direction
- If the building is heavy. Typically, *in situ* concrete frames are heavy, so the EHF should be considered

Table 9.1 provides some simple percentages of EHF to wind load, for typical floor build-ups. The breadth of the building (the width of the face with wind load applied) is not a consideration, as the wind load and EHF are both a function of this, so if you double one you double the other.

Table 9.1: Approximate percentage of EHF load

Floor-to-floor height (m)	Floor depth (m)	Timber	Steel	Concrete	Typical weight (kN/m^2)
		1	4	8	Floor weight
		2	5	10	Combined floor and wall
3	10	2	5	10	% of EHF to horizontal load, assuming 1.3kN/m^2 base pressure and C_{pe} of 1.3. (values in **bold** are at or above the 30% cut-off)
3	20	4	10	20	
3	30	6	15	**30**	
3	40	8	20	**40**	
4	10	2	4	8	
4	20	3	8	15	
4	30	5	12	23	
4	40	6	15	**30**	
5	10	2	3	6	
5	20	3	6	12	
5	30	4	9	18	
5	40	5	12	24	

Typical wind loads are roughly 1kN/m^2, so if you take a value of 1.3kN/m^2 as suggested assuming the five second guess, this allows for up to 30% EHF so under most cases the EHF can be ignored at this early stage (Table 9.1).

The EHF is a function of the vertical load. When considering load combinations, we need to decide if the wind load or vertical live load (variable action) is the leading action. When we take wind as the leading variable action, the EHF reduces as the magnitude of the vertical load reduces, due to us not taking the fully-factored vertical load, but a reduced factored vertical load (typically the factor of safety is 1.5 for the leading action, and 1.05 for the vertical variable load if it is not the leading variable action).

9.1.2 Stability design

At the concept design stage, it is essential to consider the location of the lateral stability elements, and communicate these clearly to the rest of the design team. This may not be a simple conversation, as often there is a desire for open-plan floor plates, and walls around cores often hide service risers which may clash with the stability.

The first thing is to ask the architect for a set of sketch plans. You then need to locate bays where walls/bracing could go. The obvious location is around stair and lift cores. These are ideal as they tend to run the full height of the building. However, you need to be careful with stair cores as the head room at first floor is often greater than at upper floors, therefore the depth of the stair core can change. Also, look for areas of perimeter wall where there are no windows. If these don't exist, you will need to negotiate some, or find another way of providing stability. Often, the building services engineers are also looking for riser locations, so it can make sense to work together, but be careful as vertical risers often contain large penetrations through the slab, making it harder to get the force into your stability system.

Ultimately you need a set of plans, or even better a 3D sketch that shows the locations of stability right through the building. This is often enough at the concept design stage.

Selecting the stability system

You may have already decided what type of structure you are going to have (Chapter 8). Once you have a rough idea of where your lateral stability is going to be located, you need to propose a system. There are typically three ways of stabilising a frame:

- Shear walls — concrete, masonry or timber
- Bracing — typically steel
- Moment frames — steel, concrete or timber

These approaches are summarised in Table 9.2. Shear walls are stiffer than the other systems, and are therefore more suited to taller structures, but they also use more material and provide little opportunity for windows or openings. Bracing is lighter, and with careful detailing can sit in front of windows and include penetrations, especially smaller ducts. Moment frames are very flexible and should ideally be avoided. If there are no possible locations for walls or bracing, moment frames can be utilised and can produce a very attractive finish. However, caution must be applied when using these systems. Compression/tension systems are more structurally efficient than moment systems, so a wall/braced system is preferred to a moment frame system.

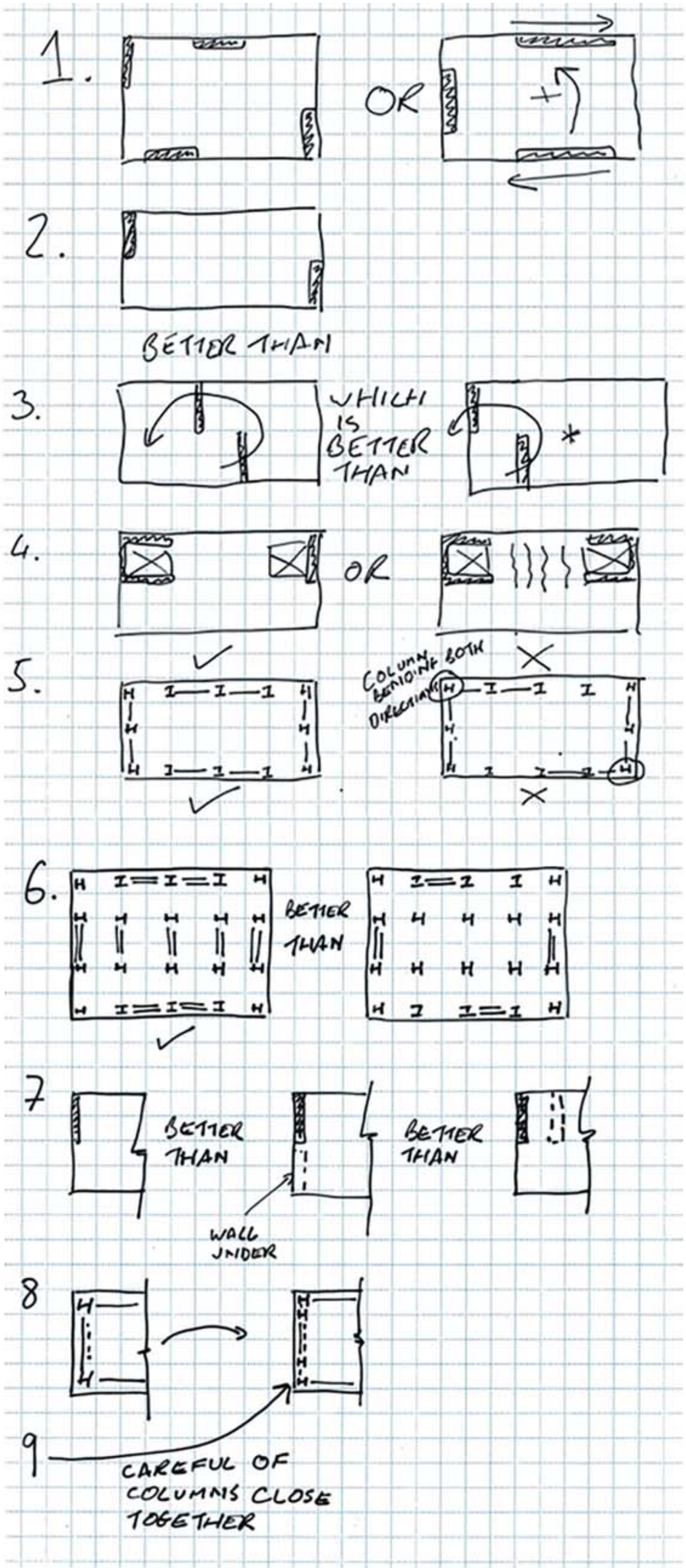
Setting out the stability system

You are hopefully now in a position to set out your lateral stability.

Here are some simple rules for stability design — the first rule is essential, the others are all 'best practice', rather than 'absolute musts'. We have all worked on projects where there is only space for a few paltry walls, in absolutely the wrong places, and you have to make it work.

For a low cost, efficient solution the tips in Figure 9.1 should be followed:

Figure 9.1: Different options for stability systems



Essential

1. For rectangular frame buildings, we need a minimum of two bays in each direction, or one bay with two bays designed to remove the associated twist (the term ‘bays’ is used here generically to mean either braced bays, shear walls or moment frames), although careful consideration of robustness is required.

Best practice

2. Bays should be located as far away as possible, to minimise the eccentricity of the load.
3. Bays should not be clumped in the centre, as this will not provide good torsional resistance.
4. Ideally avoid trapping slabs between stiff walls, as this can lead to cracking (see section on ‘Shrinkage’).
5. For moment frames, every column should be used for one frame direction only — avoid using the same column in both directions.
6. For moment frames, try to use as many consecutive frames as possible, either parallel or perpendicular.
7. Avoid changing the position of stability bays from floor to floor (see section on ‘Changing lateral stability locations’ and Figure 9.4).
8. Remember for braced frames, you need a column at either end of the bracing.
9. Try not to have too many columns near your bays, as these reduce the axial load on your bay, which helps prevent uplift in the system.

Table 9.2: Types of stability systems for different frame types

	Frame type				
	Steel	**Concrete**	**Timber**		**Masonry**
Shear wall	Concrete cores	Concrete cores and walls	Solid timber walls	Racking panels	Solid masonry walls
	OK by inspection	OK by inspection	OK by inspection — provide double the expected number for concrete	OK by inspection — provide lots of walls — suited to loadbearing wall construction	OK by inspection — provide lots of walls — suited to loadbearing wall construction
(typically)	4–20 floors	2–20 floors	2–6 floors	1–3 floors	1–2 floors[a]
Bracing	Steel bracing	N/A	Steel rods (tension)	Timber struts (compression)	N/A
	Size worst		Size worst	Size worst for connection and strut buckling	
(typically)	1–20 floors		1–4 floors	1–2 floors	
Moment frame	Steel moment frame	Concrete moment frame	Timber moment frame		N/A
	Detailed calculation for worst frame to check deflection and provisional moment connection sizing	Detailed calculation to check floor depth at column head	Detailed calculations at the earliest stage as connections will govern member sizing and careful consideration of sway stiffness to ensure limited deflection		
(typically)	1–4 floors	1–2 floors	1–2 floors		

Note: [a] Masonry is limited not by stability, but the robustness requirements for all building types except single residential dwellings, which may go up to four floors.

Diaphragms

All lateral stability systems rely on the loads being transferred from the perimeter walls and floor plates to the vertical components (Table 9.3). For floors, this is typically achieved through diaphragm action. A diaphragm is a stiff plate which is deep enough to transfer load without significant flexure occurring, so for the sake of analysis, can be assumed to be infinitely stiff. For roofs, this can also be done through diaphragm action, but this isn't always the case, so at concept design stage you may want to consider adding plan wind bracing as an alternative method.

Shrinkage

In item 4 of our best practice list, we mentioned avoiding trapping slabs between stiff walls. It is worth just pausing a little longer on this point. All materials expand and contract e.g.:

- Steel, under variations in temperature
- Timber, from changes in atmospheric moisture
- Concrete, as it cools following its initial chemical reaction

These movements, if we are not careful, can be trapped by stiff stability elements, either leading to cracking, or worse, increased stresses in members that we haven't designed for. Ideally, when setting out stability, we should avoid trapping the slab and allowing stress to build up (Figure 9.2). This may not be possible e.g. if we have lift cores at each end of the building, and require them to provide adequate strength in the other direction. If this is the case, we should consider how this stress can be relieved. For concrete structures, we can leave a section of slab and pour it 28 days later, once much of the shrinkage has occurred (although this will require careful consideration as further movement due to shrinkage, thermal effects etc will take place, especially around the construction sequence). In steel and timber structures, we can use slotted holes to enable a small degree of movement at a designated position, with the finishes detailed to accommodate this movement.

Figure 9.2: Avoiding locking in stress in the slab

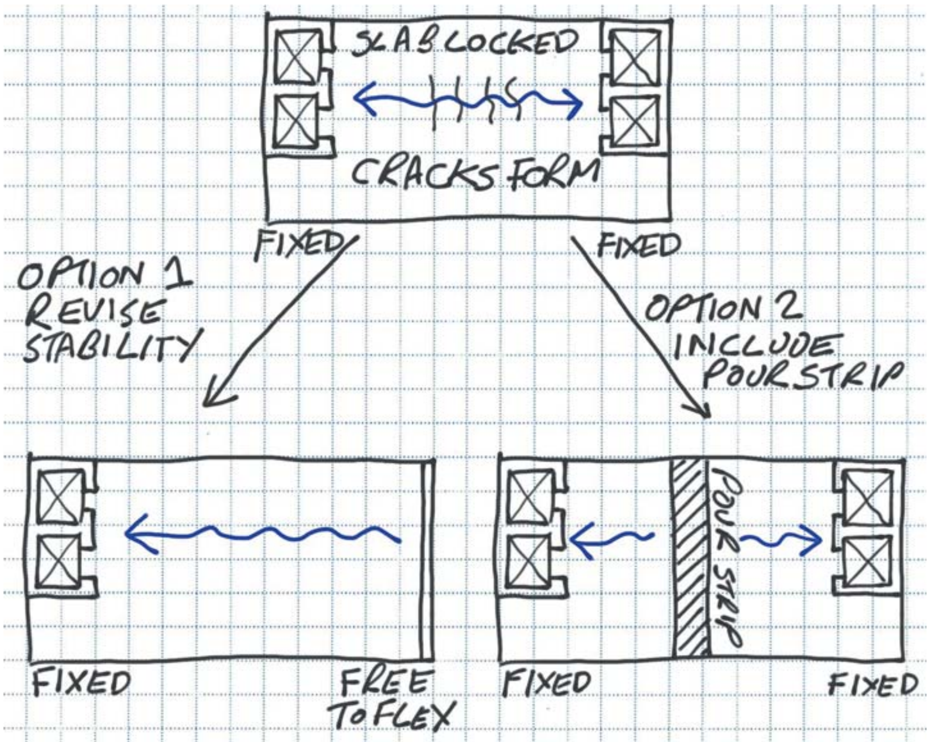


Table 9.3: Rules of thumb for spacing of different stability systems

Material	Type	Typical location	Notes	Max. spacing between lateral stability bays
Concrete	*In situ*	Roofs and slabs	Acts as diaphragm. Ideally floor plate aspect ratio is between 1:1 and 5:1	40m
	Precast biscuits with *in situ* topping	Slabs		
	Metal deck with *in situ* topping	Slabs		
	Precast planks	Slabs	Acts as diaphragm, as long as structural topping is included. Ensure topping does not have services ducts in its depth as this will limit diaphragm action	
Steel/ aluminium	Deep metal deck	Roofs	Does not typically act as diaphragm, so provide plan wind truss	40m
	Cold-rolled purlins and deck	Roofs	Does not typically act as diaphragm, so provide plan wind truss. It may be possible to use purlins as wind truss but they will need significant increase in weight/size	
Timber	CLT	Roof and slabs	Acts as diaphragm. Ideally floor plate aspect ratio is between 1:1 and 4:1. Ensure halfing joints specified to ensure shear transfer	20m
	Boarding on joists	Slabs	Acts as diaphragm. Ideally floor plate aspect ratio is between 1:1 and 2:1. It is important this is correctly detailed with board joints sitting on joist lines and nailed both sides to ensure shear transfer	10m
		Roofs	Can act as diaphragm as above. For traditional inclined truss roof more typical to provide timber in-plane bracing for wind	10m

Irregular plan buildings

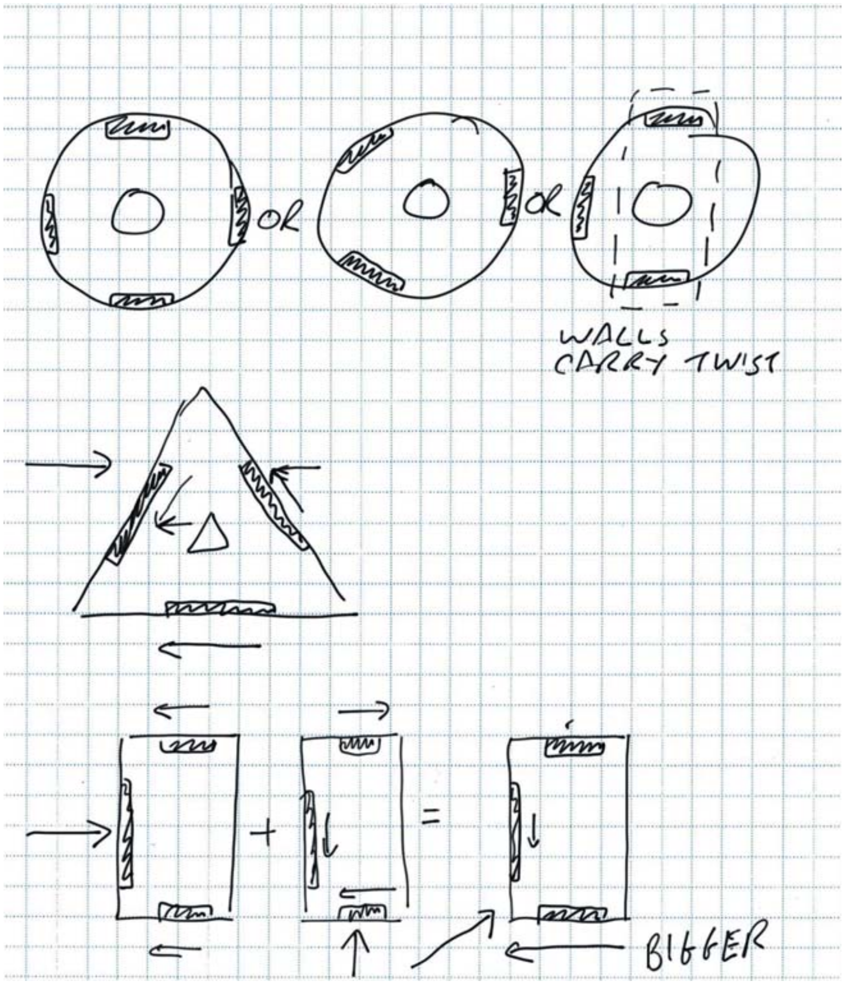
For non-rectangular buildings, you will need to think a little more carefully about stability. It is not essential that you have two walls acting in two perpendicular directions (although this is ideal). It is essential that you think about the horizontal load as acting in all directions, not just two perpendicular directions. This is also true for rectangular buildings, but the assumption is that the load from any direction can be turned into a vector in the perpendicular directions of the walls, therefore each set resists loads in that direction. However, where walls are required in the perpendicular direction to resist twist of the frame, it may well be that the worst load case is not in either orthogonal direction, but somewhere in-between e.g. the CitiCorp Center, New York[98].

Figure 9.3 shows three types of building. For circular buildings with two orthogonal sets of walls, the design approach is the same as for rectangular buildings, but with a revised pressure profile. Where non-orthogonal walls are provided, the wind needs to be considered from all 12 directions to ensure the worst load case is considered for each wall. Where walls are orthogonal but only one wall is provided in any direction, then as with rectangular buildings we need to consider the additional load from the twist as well as the significant reduction in redundancy.

For the triangular building, it can be seen that wind in the direction shown puts a load into all three walls, each one providing a vector of resistance to the wind. In this case, the largest load goes into the wall in the direction of the wind, with the other walls also taking a share and providing resistance to twisting. By just considering two directions, the maximum load on each of the three walls won't be captured.

For the rectangular building, which has an equivalent layout to the right-hand circular building, the load on the three walls for the two orthogonal directions can be seen. However, in this case, by considering a wind load at 45° to the building (the right-hand case), we can see that the load on the bottom wall goes up, and the load in the top wall reduces. This demonstrates the importance of considering multiple wind directions even if walls are orthogonal.

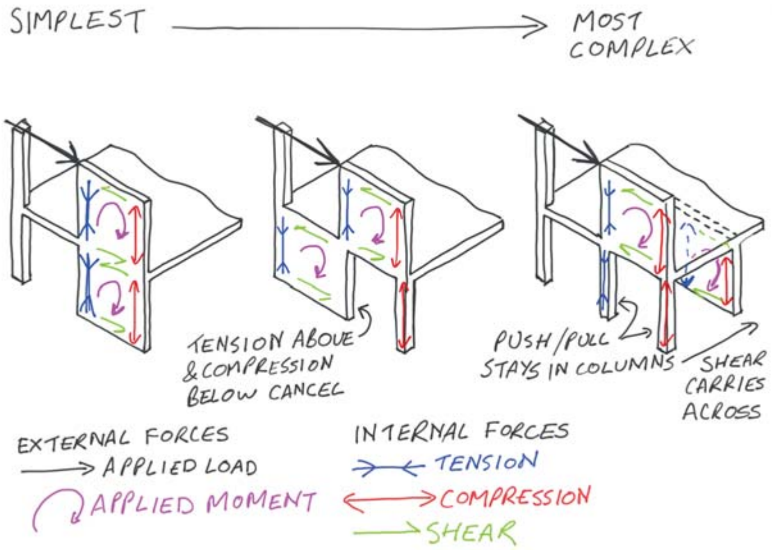
Figure 9.3: Stability solutions for non-orthogonal buildings



Changing lateral stability locations

It is not ideal to change stability locations on-plan floor-to-floor. However, sometimes there are no other options. It may be helpful to consider stability frames as cantilevers, with their base acting as a fixed support. The fixed support carries both shear and a moment. When we move a frame, the shear can be transferred (sometimes with difficulty) from one location to another. The moment however, normally in the form of a push/pull in the columns, needs to be carried all the way down to the foundations. For this reason, when a stability bay moves it is essential that the columns are maintained to ensure the push/pull is transferred to ground. For the shear, if the bay moves in the direction of the shear, it is reasonably straightforward as the centre of stiffness of the structure remains the same (middle illustration in Figure 9.4). If the bay moves perpendicular to the bay, the centre of stiffness shifts, creating an eccentricity in load and a potential twist. The slab will need to carry this force, which can sometimes prove problematic (right-hand illustration in Fig. 9.4).

Figure 9.4: Effect of moving stability locations floor-to-floor



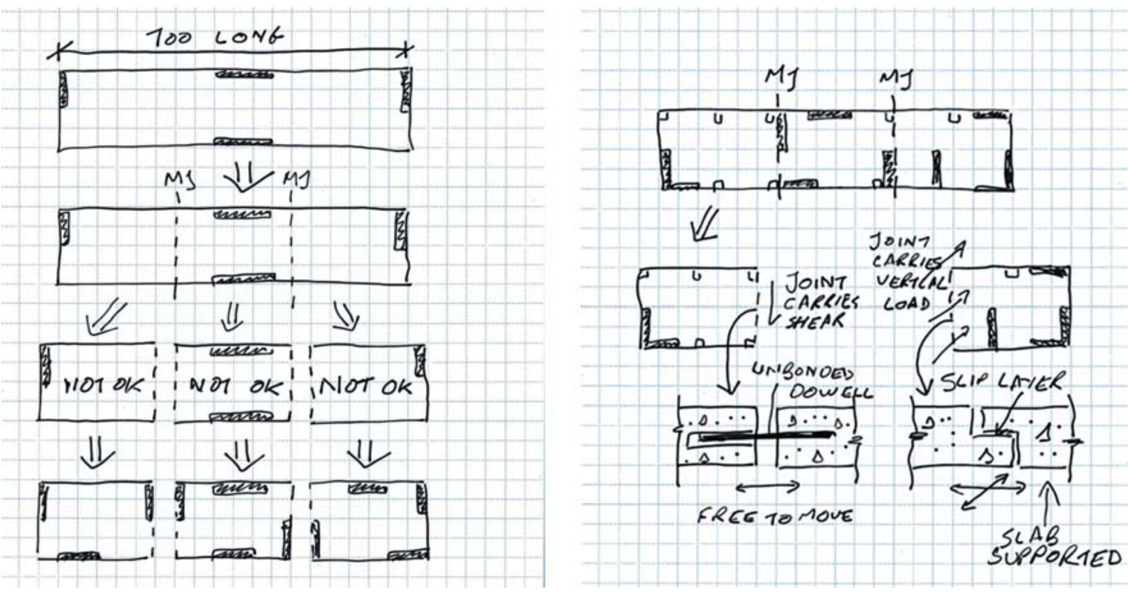
Structural movement joints

For large-plan buildings, where there is a significant change in building height or variable ground conditions, it may be necessary to introduce structural movement joints (Figure 9.5). These split the building into smaller parts, and prevent excessive build-up of stress due to thermal movement and shrinkage. Ideally, each section of the building should be treated as an individual building, and lateral stability should be provided in both directions for each section. This will typically lead to double columns at the interface where there is a movement joint.

Sometimes it is not possible to completely separate the sections — there are two typical reasons for this:

- It is not possible to provide stability for all portions in all directions. In this case, the shear may need to be carried in the short direction (we are trying to separate the floor plates in the long direction). This can be done through the use of dowelled connections
- Double columns are not architecturally acceptable – in this case a sliding joint will be required. This can be achieved in the same way or through a halving joint which will enable movement in both directions but provides vertical support

Figure 9.5: Strategies for stabilising structures with movement joints

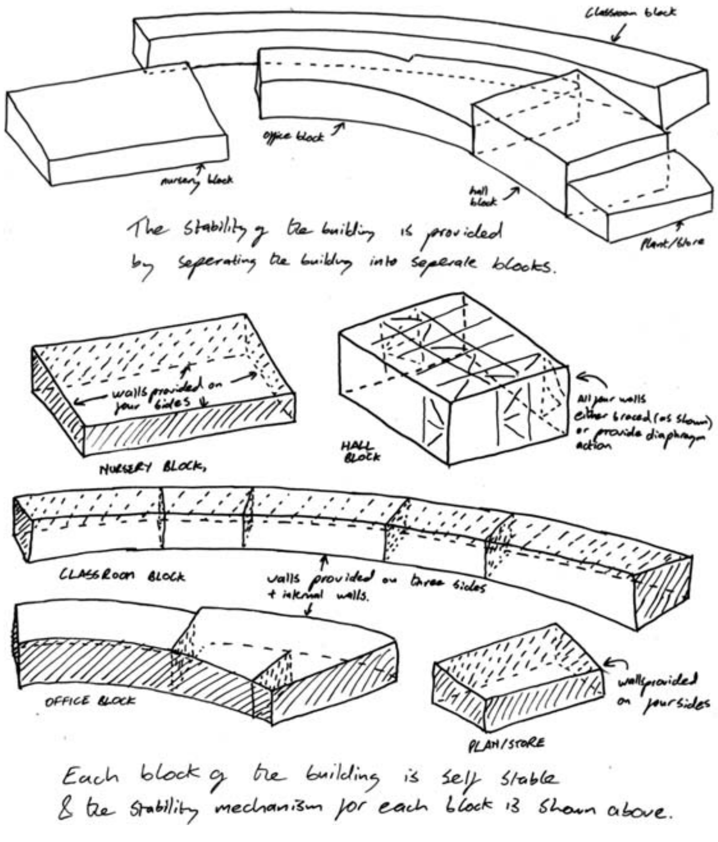


It is important, before finalising stability, to agree movement joints with the architect (they will need to be reflected through the facade, so the location is key), as they will affect the location of your stability walls.

For masonry facades (brick, block or stone), we often require movement joints for movement, due to thermal and moisture effects. While these should match the structural movement joints, they are required at much closer centres (typically 6m for blockwork and 12m for brickwork), and are typically set out by the architect with input from the engineer. If we are designing a steel or concrete frame with a masonry facade, we do not need movement joints in the frame at every point where the masonry movement joints occur in the facade.

Drawing the solution
At this stage, you should have a stability scheme and be ready to communicate it. The best way to show a stability solution is to draw a 3D frame and show the location of walls, bracing and moment frames (Figure 9.6). If there are movement joints, consider exploding the diagram, so that you can convince yourself of the adequacy of each element. While an experienced engineer may be able to understand and appreciate the stability by simply marking it onto plans, the complexity of stability means that a 3D drawing is more likely to highlight challenges e.g. changes in location.

Figure 9.6: Stability scheme



Checking global design for overturning and uplift
Having drawn the stability system, we now want to make sure we have considered overturning and uplift at the base. Uplift occurs where the tension force from the moment exceeds the dead load of the building acting on that stability element.

While a detailed analysis will not be required for all walls/braced bays, walls with a high aspect ratio (tall and not very long/wide apart) would benefit from further consideration at this stage. There are two simple questions to ask:

1. Do we anticipate uplift?
2. If we do, what mechanism will hold it down?

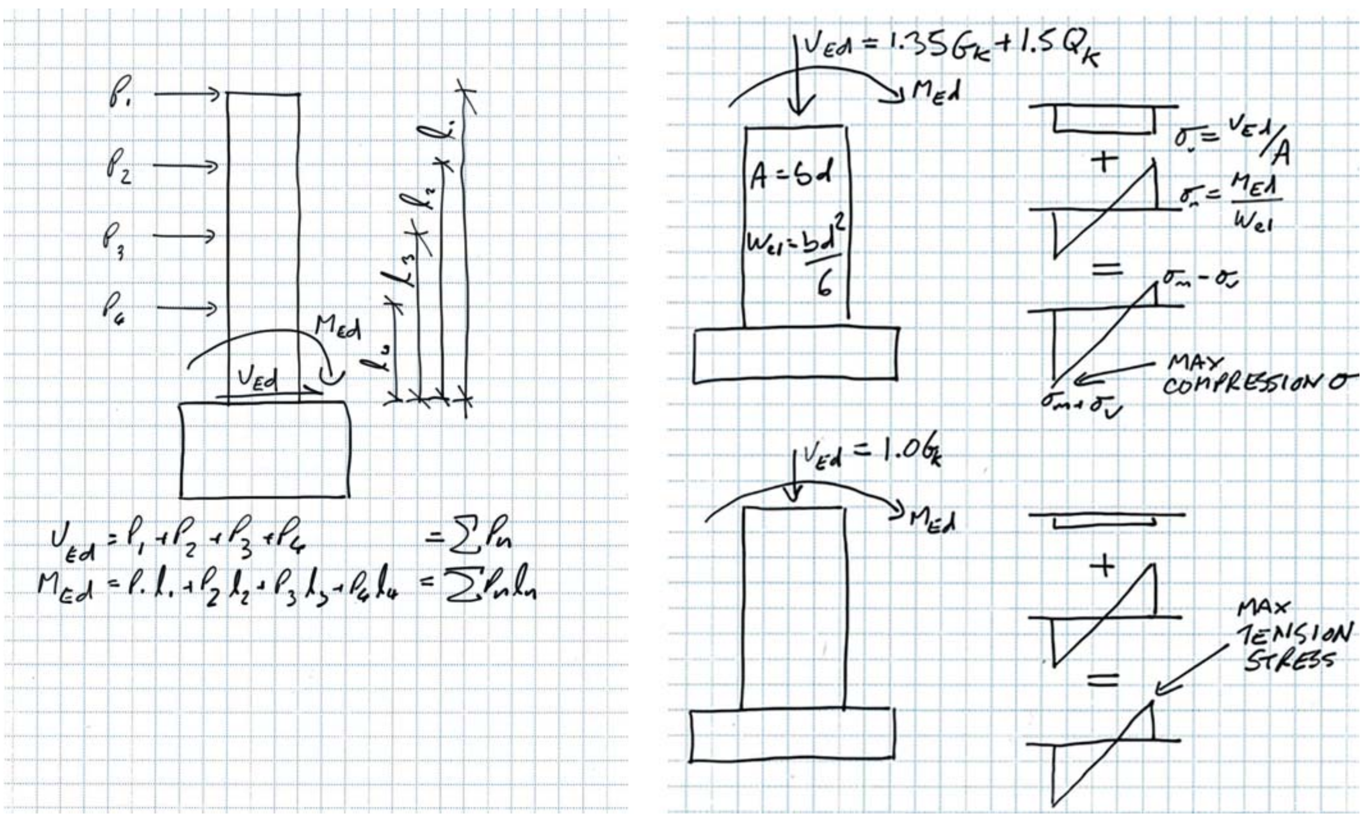
Ideally, we want to avoid uplift altogether, and at this stage this should ideally be our strategy. A quick analysis of the moments, shear and vertical load (Figure 9.7) will reveal whether uplift is a concern. If we think we have uplift, we should push the ends of the wall or columns further apart, if the architecture will allow. This has two effects:

- Increases the lever arm, reducing the uplift force
- Leads to greater loads on the columns/stability elements, helping to resist the uplift

If we can't do this, we need to ask what will hold down our building? If we have pads and footings, we will need mass concrete. Every cube of concrete provides 24kN of resistance, so if you need to overcome 1000kN of uplift, that will be more than 40m^3 of concrete — or seven trucks full!

If we have piled foundations, we will need to add tension piles. Unlike compression piles, tension piles cannot utilise end-bearing, so work on skin-friction only. We can anchor them into rock, but this becomes expensive. The capacity of tension piles can vary significantly from zero to roughly the same as compression piles. It is not possible to estimate the tension pile capacity until you have carried out the ground conditions, and the more accurate your information, the better your tension pile estimate will be.

Figure 9.7: Calculating shear, moments and stresses on shear walls



9.2 Element design (and when you can ignore it)

Having chosen your stability system and checked it for global overturning, it may be necessary to size some elements (Table 9.4). This should only be done near the end of the process, once the design team is confident about the location of stability bays. At concept design, you may only need to do a simple height-to-width check, or you may have to do a full detailed design.

If in doubt, or if you exceed the rules of thumb in Table 9.4, it is always better to do more design work now, and avoid a design that you are not confident works, or requires structural gymnastics to prove it stands up later.

Table 9.4: A quick guide to stability element design

Element	Level of design at scheme design	Rule of thumb/points to note
Concrete shear wall	Draw, ensure walls are adequate and check rule of thumb	Walls to have height-to-length of min. 4:1 (or equivalent stiffness if lift/stair cores). Typical thickness is 200mm but check cover for fire
Masonry shear wall	Draw, ensure walls are adequate and check rule of thumb	Walls to have height-to-length of min. 2:1. If large openings occur consider a steel picture frame
Timber shear wall	Draw, ensure walls are adequate and check rule of thumb	Walls to have height-to-length of min. 2:1
Steel bracing	Draw, ensure braced bays are adequate and size most highly loaded	Ensure bracing can fit within wall build-up if not expressed
Timber bracing	Draw, ensure adequate number of braced bays are provided and size most highly loaded in both tension (connection check) and compression (buckling check)	Connection design may govern bracing size and/or size of vertical elements bracing is connected to
Steel moment frame	Draw, ensure adequate number of moment frames are provided, size most highly loaded and check deflection/ensure you clearly draw and label moment connections	Moment connections will have an impact on cost and ensuring they are all clearly labelled on drawings will avoid problems later. Note that connections may be deeper than main sections so the architect should be advised
Timber moment frame	Draw, ensure adequate number of moment frames are provided and size most highly loaded, paying particular attention to the connection size which will most likely govern the size of both beams and columns. Check deflection	Ensure all moment connections are drawn on GAs

9.3 Robust design and the pitfalls to be avoided

In addition to lateral stability, the other areas of robust design that need to be considered are:

- Ductility
- Redundancy
- Insensitivity
- Uncertainty
- Fire

Ductility
Ductile designs are always preferred to brittle ones, as they provide warning signs of failure and often enable redistribution of load, if the assumed structural model does not match the built reality. Most major building materials (steel, concrete and, arguably, timber) can be designed to ensure they act in a ductile manner. However, some more unusual materials e.g. glass, ceramics and fibre reinforced polymers may act in a brittle way. In these cases, a much higher level of care is required, and simply increasing sizes of sections if we are not sure about the magnitude of stresses will not be enough and, in some cases, may prove disastrous as the increased stiffness will attract more load, which may lead to sudden brittle failure.

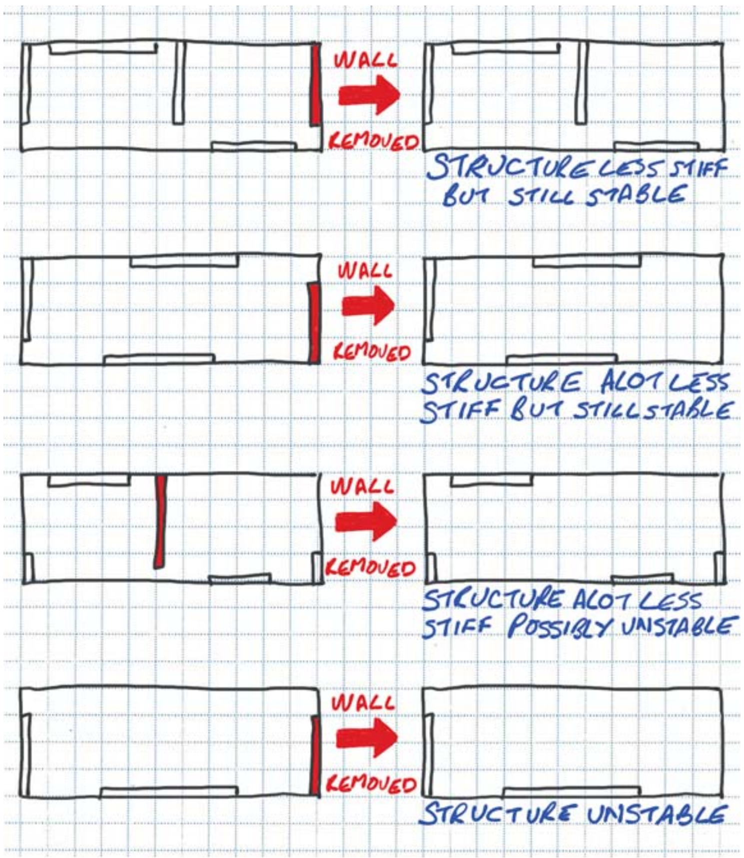
Redundancy

The concept of redundancy is that if a member were to fail, there is another load path which can be used. It is the gold standard of robust design, but is not always possible. A typical structure with no redundancy is a cantilever which, when the support becomes a hinge, turns into a mechanism. There is no alternative load path or catenary action that can be achieved. Another example is a truss, as, if one element were to fail, it will typically not have an alternative load path, and the whole system will fail. This does not mean that we cannot use a cantilever or truss, but they are less robust, and therefore require careful consideration.

The same is true of stability systems. Figure 9.8 shows a number of different layouts. We can consider the redundancy, by removing each wall in turn and seeing the impact. When we have more than two walls or braced bays acting in the same direction, the loss of one wall will typically lead to a loss of stiffness, but the structure remains stable. When we only have two walls or braced bays acting in each direction, the loss of one wall will lead to a large reduction in stiffness, but the system remains stable, as the orthogonal walls will still act to resist the twist. Sometimes, when we design a stability system we find that one wall or braced bay carries almost all the load, with the other walls or braced bays just removing the eccentric load. In this case, we are very reliant on this wall and removal will lead to both a huge loss of stiffness and possibly an unstable structure, depending on the adequacy of the remaining walls. We may only be able to provide three walls due to other constraints, often architectural ones. In this case, the loss of a wall would lead to a complete loss of stiffness, and the system will become unstable.

We must go through these thought processes, as we consider the redundancy of our stability system. If we can only provide three walls, it is important to recognise the lack of redundancy, and ideally we should try and provide more than two walls of similar stiffness, especially in the dominant wind load direction (most buildings are rectangular on-plan, therefore the wind load is likely to be larger when acting on the long face). The aim is to make the structure efficient (so not adding walls unnecessarily) but also robust with, ideally, a good level of redundancy.

Figure 9.8: Redundancy in stability systems



Insensitivity
A robust structure is insensitive. This often (but not always) means that small changes in construction and/or use, do not alter the structural performance.

Most modern structural forms are insensitive to minor changes, whether ground movement, shrinkage or thermal movement. However, some are very sensitive to small movements. A typical example is a masonry arch, especially flat arches, where ground movement can lead to small horizontal movements, which can move the thrust line in the arch leading to cracking and, in some circumstances, collapse. Other arching structures, such as grid shells and large steel arches, can be similarly sensitive to small movements, and these need to be carefully considered.

Precast connections, which have very small bearing areas, can be highly sensitive to small changes. These should be avoided, and reasonable bearing areas provided.

Uncertainty
Designing for uncertainty is an essential part of the design process. Many things are unknown as we design our building:

- What will the loads really be?
- What will the material strength really be?
- Will our building be truly built to plumb?

Actually, many of these unknowns are, in fact, well known and documented, and are already accounted for in the design, both by using characteristic values (rather than mean values), and by factors of safety. We certainly don't need to make a further allowance for them in the design.

Other uncertainties arise from site specific constraints e.g whether we might find 'gulls' in the ground (large voids that can occur in limestone rock). Sometimes, these are completely unpredictable, but often we can make an allowance for them in our design e.g. by designing strip foundations to span over them.

Foreseeing these unknowns is a skill of the designer, and comes from a combination of asking great questions and experience. The trick is not to design a building that can withstand every eventuality, but to devise a design which is adaptable and can be quickly and easily modified, as and when the uncertainties are discovered. In the 'gulls' situation for example, if we had used pad footings, we may have struggled to quickly react to discovering them, whereas strip footings just require the addition of a rebar cage, and would span over the gull, avoiding delays on site.

Fire (with input from Jonathan Hall of Buro Happold)
Designing for fire is a complex issue, made more confusing by the variation of scope for different members of the design team from project to project. Typically, the scope will be as follows but this will need clarifying on every project you work on. Most projects, particularly larger projects, will have a project fire engineer. For smaller projects in the UK, the architect may take ownership of meeting Approved Document B of the Building Regulations[98].

- **Fire engineer**
  Sets performance criteria for the design to meet Building Regulations requirements for fire safety:
  - Confirms structural fire resistance and required compartmentation within the building
  - Confirms the requirements for fire safety systems e.g. detection, suppression, smoke control
  - Confirms the travel distance and occupant capacity limits for the building
  - Highlights the criteria for internal and external linings
  - Advises on external fire spread and firefighter access, to and within the building
- **Architect**
  - Confirms safe egress in line with the criteria specified by the fire engineer
  - Confirms how fire rating and details for fire compartments are achieved
  - Ensures strategy for limiting spread of fire (including, but not limited to, specifying finishes on exposed timber)

- **Building services engineer**
  - Confirms smoke ventilation strategy, if deemed necessary by the fire engineer, especially if mechanical forcing is involved
  - Ensures mechanical systems are designed to prevent spread of fire and smoke
- **Structural engineer**
  - Ensures structure remains sound for duration of the fire resistance period
  - Supports architect in ensuring structural materials provide adequate fire compartmentation
  - If providing facade engineering (which is typically a separate appointment to structural engineering), ensures facade is designed to prevent spread of fire

At concept design, the engineer should be asking these two questions, as a minimum:

1. What is the fire resistance requirement for the structure, and for different spaces in this building?
2. How is the fire resistance achieved?

For exposed structure, this can be even more challenging. Table 9.5 gives some typical strategies.

Table 9.5: Fire-proofing strategy for exposed structure

Element material	Protection type/strategy	Fire resistance[a]	Comments
Concrete	Provide adequate cover for rebar	30–240 mins	
Steel	Intumescent paint	30–120 mins (up to 240 mins with specialised products)	Requires maintenance. Check visual appearance with architect
	Exposed steel fire designed using concrete infill	30–60 mins (up to 180 mins with concrete reinforcement if designed as a composite member to BS EN 1994-1-2[100])	Requires very careful consideration and specialist design advice
Timber — joists, beams and columns	Charring of exposed timber	0–30 mins	Requires design for fire using reduced charred section. Requires flame flame-retardant[b]
Timber— solid timber such as unidirectional laminated timber (ULT) and CLT	Charring of exposed timber	30–60 mins	Requires design for fire using reduced charred section. Requires flame-retardant coating[b]

Notes:
[a] Fire resistance period may be reduced by use of mechanical restraint systems such as sprinklers. This is typically specified by the project fire engineer.
[b] There is a debate about use of timber for structure and facade of medium- and high-rise buildings, so latest guidance should be consulted before advising further.

9.4 A brief guide to disproportionate collapse and when it needs to be considered

Disproportionate collapse is where the *degree* of failure is considered disproportionate to the *cause* of failure.

Building Regulations Approved Document A categorises buildings into four classes (1, 2A, 2B, and 3). Depending on the class, the requirements of disproportionate collapse vary. Reference should be made to the Building Regulations for the full definition of the classes. Some typical cases are summarised in Table 9.6 (which is an abbreviated version of Table 4.1 in the IStructE *Practical guide to structural robustness and disproportionate collapse in buildings*[92]).

Table 9.6: Definition of building class for typical building types

Building type	Building class			
	1	2A	2B	3
Agricultural	All			
Houses	1–4 storeys	5 storeys (single occupancy)	≥6 storeys	
Hotels, flats and other residential buildings		1–4 storeys	5–15 storeys	≥16 storeys
Offices		1–4 storeys	5–15 storeys	≥16 storeys
Retail		1–3 storeys and <2000m^2 floor area per storey	4–15 storeys and <2000m^2 floor area per storey	≥16 storeys or >2000m^2 floor area per storey
Building to which public are admitted		1–2 storeys and <2000m^2 floor area per storey	≥3 storeys and <5000m^2 floor area per storey	≥3 storeys and >5000m^2 floor area per storey
Educational		1 storey	2–15 storeys	≥16 storeys

At scheme design stage, it is important to consider disproportionate collapse to ensure a feasible approach is available. Full detailed design can then be carried out at a later stage. Table 9.7 provides typical approaches for different building classes and different construction methods. They are based on the recommendations of Section 9 of the IStructE guide.

Under Class 2B, there are often multiple agreed strategies — from element removal to providing horizontal and vertical ties. We are required to provide at least one of these in our design. At concept stage we should consider which strategy would be most appropriate, and record our approach and any critical details that should be considered. If we choose to use the 'key element' approach, it is important to do some initial checks, to ensure that the elements that we are treating as 'key' are suitably sized, and to make a record. This will be particularly important on the Stage 2 drawings.

Table 9.7: Approaches to disproportionate collapse

Construction type		Building class 1	2A	2B	3
Steel frame		No additional considerations	Provide horizontal ties	Provide horizontal and vertical ties	Carry out a risk assessment but as a minimum provide the same as 2B
				Or notional element removal	
				Or key element design	
Concrete frame	*In situ*	No additional considerations	Rebar continuity typically provides adequate ties	Rebar continuity typically provides adequate ties	Carry out a risk assessment but as a minimum provide the same as 2B
				Or notional element removal	
				Or key element design	
	Precast		Ensure connections provide adequate horizontal ties	Ensure connections provide adequate horizontal and vertical ties (vertical ties can be hard to form)	
Timber	Loadbearing stud and joist floor (platform timber frame)	No additional considerations	Typically, connections provide requirements for horizontal ties and no further consideration is required at scheme design	Provide rim beam, and sequentially check wall removal	Carry out a risk assessment but as a minimum provide the same as 2B
	Solid timber (CLT and unidirectional laminated timber (ULT))			Ensure solid wall panels can act as beams, and connections can carry hanging floor load	
	Frame (sawn timber/glulam/ laminated veneer lumber (LVL))		Provide horizontal ties	Horizontal and vertical ties	
				Or notional element removal	
				Or key element design	
Masonry		No additional considerations	Provide horizontal ties	Not recommended. Provide horizontal and vertical ties — in practice this is very difficult to achieve	Not recommended