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3.2.5 Investigation parameters and methodology

Where deep geotechnical investigation is to be undertaken, the following guidance applies with reference to section 13 of MBIE's Residential Guidance (noting that many industrial building sites have soil profiles equivalent to residential TC3 land):

› the geotechnical investigations should be determined and overseen by a CPEng Geotechnical Engineer competent in geotechnical earthquake engineering

› industrial buildings have a wide range of plan areas and loading conditions and the number and depth of tests required will vary accordingly. The extent of testing required will be subject to the judgement of the geotechnical engineer according to the variations in soil profile found across the site and the scale and type of structure planned. The number and depth of tests needed may also be informed by the number and proximity of existing deeper information in the area available on the Canterbury Geotechnical Database

› the depth of testing is subject to the discretion of the geotechnical engineer involved. In general, a depth of 15m will encompass the extent of the most damaging liquefaction, however with the larger building size and generally higher building loads, it may be prudent for most industrial buildings to extend some tests to greater depths to be able to properly assess a piled foundation option. The early termination of tests, cone penetration tests (CPT) in particular, may result in the loss of potentially useful information regarding possible pile founding depths, ground improvement options, overall site settlements and general site characterisation

› CPT tests are usually suitable for the type of site being considered and are likely to be the predominant investigation test, and are preferred to borehole standard penetration tests (SPTs) in determining liquefaction susceptibility. CPT equipment should be calibrated, and procedures carried out to ASTM D5778-12. Where SPTs are used, it is important that the equipment is properly energy rated so that an appropriate energy ratio can be used to correct the SPT 'N' values

› the MBIE Residential Guidance requires a standard liquefaction analysis methodology in order to obtain liquefaction settlement index numbers related to the Technical Category (TC) classes. Non-residential land has not been zoned in TC classes, and equivalent foundation options have not been developed for industrial buildings. Consequently, there is not the same need to adopt a standard analysis methodology. However, there is merit in using the standard method as it then links the site into the broad damage categories of the MBIE Residential Guidance and may provide useful parallels to expected foundation performance and foundation systems of residential buildings. The recommended method is to use report UCD/GCM-14/01 "CPT and SPT Based Liquefaction Triggering Procedures" by R Boulanger and I Idriss (2014) (available at http://cgm.engr.ucdavis.edu/library/reports/) ensuring that the following requirements are met:

  - at SLS for sites in the Canterbury earthquake region, both the M7.5 / 0.13g and a M6 / 0.19g design case should be analysed (and the highest calculated total volumetric strain from either scenario adopted)
  - at ULS it may be sufficient to simply analyse the M7.5 / 0.35g case for sites in the Canterbury earthquake region.

If fines contents are being derived from CPT data, the new FC / Ic relationship in the 2014 methodology should be adopted. A CFC fitting parameter of 0.0 should be used, unless appropriate lab data or other evidence supports a different value. For example, Robinson et al (2013) suggests a value of CFC = - 0.07 could be adopted for liquefiable soils along the Avon River. Refer to 'Clarifications and Updates to the Guidance', Issue 7 October 2014, Question and Answer 50 and 51 for more detail. Only data obtained directly from CPT or SPT measurements should be used in carrying out liquefaction assessments

› ground motion inputs for SLS and ULS liquefaction analysis for deep soft soil (Class D), for IL2 building sites are:
  - SLS 0.13g at M 7.5, and 0.19g at M6
  - ULS 0.35g at M7.5

Further information on these peak ground accelerations is found in Appendix C2 of the MBIE Residential Guidance and in Question and Answer 50.

3.3 Structural considerations

Although every building should be considered on its own merits, there are some trends in performance that have been observed, many of which relate to particular structural characteristics of the buildings.

The following is a summary list of issues that may be considered, some of which are addressed in more detail in the following topics:

› differential settlement - local versus global settlement:
  - effect of significant variations in bearing pressure under foundations
  - flexibility and effect of differential settlements
  - panel connection distress.

› roof bracing issues (or lack of roof bracing)

› stiffness compatibility issues

› non-ductile or brittle behaviour

› large building effects (pre-existing issues with regard to temperature or shrinkage movement)

› load path issues

› poor distribution of mass/strength

› reduced functionality of floor and foundation systems.

3.3.1 Primary structure

The primary structural systems of industrial buildings have generally performed adequately. As single storey structures, the seismic actions are generally relatively low and frequently earthquake loading is not the governing load case. This has been an important factor in many older structures that may not have been designed for seismic load. Even where the direct seismic loads have significantly exceeded the original design load, the buildings have had sufficient reserve capacity. In addition, the ductility demand is usually low.

Hence, many of the problems that have been observed with the primary structure are the result of differential settlement and compatibility issues, rather than direct shaking related outcomes.

A detailed methodology for the assessment of damage to the primary structure is presented in section 3.4 of this document.

Tension-only bracing is an exception where the actual ductility demand may have significantly exceeded the system capacity, particularly in older buildings. In such cases, the bracing is frequently poorly detailed by current standards. The outcomes in such cases may have been fractured bracing or connections.

Tension-only systems may also have performed poorly where there has been differential settlement, as this adds significantly to the ductility demand. This should be given careful consideration when repairing these systems, noting that it is not always easy to add ductility, and that differential settlement will generally result in the bracing in one direction going slack while the other direction ductility demand may be exceeded.

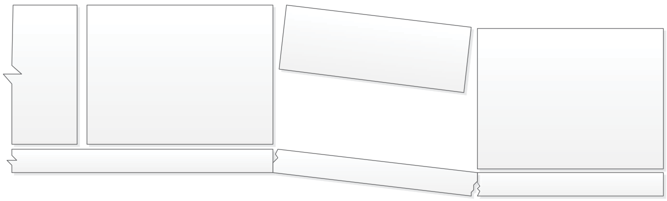
Unless specifically detailed for the effects of overstrength actions, tension-only bracing should be treated as elastic or nominally ductile for the purposes of assessment. Care should be taken to include the C_s factor in assessing demand, in accordance with NZS 3404.

3.3.2 Differential settlement effects

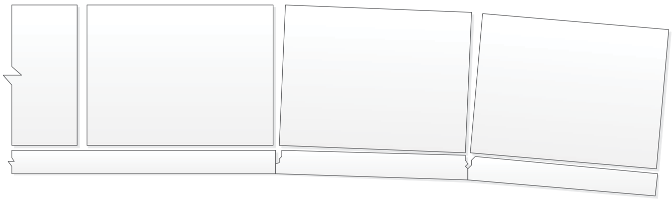
Where the strength and stiffness of the wall and foundation assembly are high enough, the soil deformation may be normalised – that is, the resulting differential settlement will have resulted in a constant slope along the line of the wall(s). However, where there are significant strength and/or stiffness changes in the wall and foundation assembly, the differential settlement may be concentrated in a discrete location, resulting in damage to adjacent elements of the structure. This is illustrated in Figure 3.4, particularly items b, c, d, and e.

Figure 3.4: Influence of wall and foundation stiffness on settlement

a) Wall and foundation stiffness and connection strength result in constant slope.



b) Strength and stiffness discontinuity due to opening results in abrupt slope change.



c) Change in slope may indicate overstressing of panel connections.

d) Change in slope may indicate overstressing of panel connections.

e) Offset may indicate overstressing of panel connections.

A similar outcome may also result when panel connections fail or are highly flexible. Refer to section 3.3.4 of this document for more detail.

3.3.3 Roof bracing

3.3.3.1 Bracing performance

Roof bracing in industrial buildings generally consists of steel angle or rod diagonal braces often with double purlins or heavier steel elements to act as collectors and to resist the increased loads imparted by the bracing. The location of braced bays is variable, but they are often located in the end bays, adjacent to the end walls, where most of the wind or seismic load is applied.

Roof bracing (and in some cases diagonal tension bracing in walls) has failed due to earthquake action in a number of cases. The requirements for bracing and bracing connections in the pre-1976 New Zealand Standards did not reflect the demand on these systems. Many bracing elements have simply been overwhelmed due to the extremely high demand caused by shaking in excess of the design load. However, there are cases of premature failure where ductility has been assumed in excess of what the brace or its connections can reasonably tolerate.

It is also possible, in some cases, that excessive demand has resulted from differential settlement where the strength of the brace or its connections has not been enough to impose a uniform displacement on the foundations.

The implications of bracing failure have rarely resulted in a life safety hazard, as these systems are typically located in single storey structures with low gravity loads. However assessors should be careful to consider whether the failure results from direct shaking or is settlement induced. If the former, simple comparable replacement of the brace may be appropriate but if the failure results from differential settlement effects, a different approach may be required.

More commonly, the bracing may have yielded or loosened, resulting in increasing drifts and/or differential settlement. This may result in damage to non-structural elements but may also result in alternative load paths developing, often in non-structural elements (such as lightweight cladding systems) or in weak axis bending and shear of portal frames. The performance of those latter systems may have been compromised by the distortions imposed or the reduction in support provided.

3.3.3.2 Absent bracing

A number of industrial buildings have no roof bracing, by design. In these cases, lateral stability is generally provided by cantilevered columns and walls in one or more directions.

Observed issues with industrial buildings constructed to the 'no roof bracing' design approach include:

› where the foundations are on soft or liquefiable ground, excessive deflections of the cantilevering columns may have resulted in significantly greater displacements in the superstructure than the designer anticipated, resulting in increased damage to other elements supported by these elements

› although the designated systems may have sufficient strength, the flexibility of the system may result in displacement that exceeds the deformation capacity of the roof cladding. Consequently, the roofing has acted as a structural diaphragm and taken the initial load, resulting in overstressed connections and other non-structural damage, typically manifesting as tearing of the sheeting.

3.3.4 Out-of-plane failures

Failure under face loading of concrete panels, concrete block walls, or unreinforced masonry walls has often been observed. This may have resulted either from the level of shaking being higher than expected, or from under-design of the wall elements. In either case, the matters for consideration now include:

› the ongoing ability of the panel to continue to resist face loads in the future

› the extent to which the panels support gravity loads at high level beyond their own self weight

› the required repairs to the panel to restore structural integrity.

It is important to determine the type and form of reinforcement in the panel, particularly at connections. This may vary according to the age and construction type.

In areas where panels have moved out-of-plane, the adequacy of the roof supporting members to provide continual gravity load support should be checked.

3.3.4.1 Reinforced concrete (RC) panels

Reinforced concrete (RC) panels are probably the most common form of exterior wall on industrial buildings. They often are required to resist in-plane loads as well as to act as cladding. Since the 1950s, RC walls have often been constructed using tilt up techniques, but have also been cast off-site and trucked to site for erection using similar methods.

A feature of many RC walls in Canterbury has been the use of cold-drawn welded wire mesh as reinforcement, apparently in some cases right up to the time of the earthquakes, notwithstanding limitations contained in the materials design standard, NZS 3101:2006 clause 5.3.2.6. This steel is generally not capable of developing any significant strain beyond yield. Ductile mesh has only been available in the New Zealand market since 2011. Furthermore, this reinforcement is often relatively light, reflecting the low demand anticipated at the time of design, which was often governed by lifting considerations for tilt panels.

As the assessed demand under face loading may now be significantly greater than when the panel was designed, it is likely that some panels have inadequate reinforcement to resist even 33% of current code demand (in order to satisfy earthquake-prone building criteria). In these cases, it is also possible that the flexural capacity of the panel reinforcement may be less than the cracking moment of the panels.

Because the mesh is brittle, the implication of almost any level of inelastic displacement of the panel is that the mesh may fracture. Fracture or necking of mesh has been observed in other situations at crack widths of as little as 2mm.

This is dependent on the bond of the mesh to the concrete but even though the mesh is not deformed, it is not safe to rely on the bond being broken over the unanchored mesh wire length, typically 150mm between cross wires.

Panel aspect ratios have been progressively reduced over the years as construction methods and code changes have enabled thinner panels. A study in 2005[8] quoted H/t ratios in excess of 70. Although these walls have apparently performed adequately for in-plane loading, very high deflections under face loading may result in significant P-δ effects, which may not have been allowed for in design. Some cases of excessive permanent deflections have been observed.

8 Poole, R.A. (2005). Report to Department of Building and Housing – Review of Design and Construction of Slender Precast Walls. Wellington, New Zealand: Department of Building and Housing.

Assessment and repair of panel buildings has been previously addressed in more detail in section 9C of the Detailed Engineering Evaluation Technical Guidance[9]. This section is reproduced in Appendix C5 of this document.

3.3.4.2 Concrete block walls

Major factors influencing the behaviour of concrete block walls are the extent of filling of the walls and the distribution of reinforcement in the walls. It can also be important to separate the design intent from the actual work as performed on site, as there have been many observed instances of the as-built construction not matching the available drawings.

Older concrete block construction was frequently unfilled or only partially filled. Use of bond beams only at the tops of unfilled block walls in these older construction examples is common. Partially filled walls are also common, with the filled cores being lightly reinforced, extending to bond beams. The bond beams are generally at the top of the wall, but sometimes also at intermediate levels. There were several observed instances of failed walls showing up construction defects whereby the filling and the reinforcement were in different cores.

Aspect ratios of block walls are generally not as high as for thin precast concrete wall panels and so additional P-δ effects are not generally a significant concern in concrete block walls.

3.3.4.3 Unreinforced masonry walls

Seismic behaviour of unreinforced masonry under face loading is generally poor. Unless there is sufficient confinement, masonry tends to topple at relatively low loads, particularly under cyclic loading where the mortar cracks and tensile capacity reduces to zero. This is more pronounced in light axial load situations, eg in the uppermost storeys of buildings or in infill walls where the frame carries the weight of the infill above.

Most unreinforced masonry walls in industrial buildings are brick, with larger structures often featuring infill panels within concrete frames. The degree of confinement offered by the concrete may provide a significant strengthening effect for out-of-plane actions where there is direct contact. However, this may also cause a short column effect in the frame under in-plane action. Guidance is given for analysing this in the NZSEE Guidelines[10].

3.3.5 Assessing connections

Careful inspection for damage to precast panel connections is required. Typically damage is concentrated at connections and some connections may have been compromised by the earthquakes and require replacement.

All panel connections require consideration but, in general, will fall into two categories:

1. primary panel connections
2. other panel connections.

Aspects of each are discussed in the following topics.

9 MBIE, Engineering Advisory Group. (2006). Draft Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 3 Technical Guidance. Retrieved from www.sesoc.org.nz/images/DEE_Part%203_Technical_Guide_S8_R4.pdf

10 New Zealand Society for Earthquake Engineering Inc. (2006). Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. Retrieved from www.nzsee.org.nz/db/PUBS/2006AISBEGUIDELINESCorr3_(incl_2014_updates).pdf

3.3.5.1 Primary panel connections

The primary panel connections are vulnerable in all cases where there are stiff panels that form part or all of the primary lateral load resisting mechanism and where discrete connections are transferring loads (as opposed to other jointing systems that result in fully integrated walls or frames). In this case, the distinction is twofold:

1. the connections may become the focus of imposed displacements because of the relative in-plane size and stiffness of the panel elements
2. the connection itself may be the determining factor in the capacity of the overall lateral load resisting system.

For industrial buildings, this type of connection mainly relates to precast concrete panels, but the same considerations may apply to other connections between primary elements. The connections may be either brittle or flexible/ductile and this may have considerable influence on the behaviour of the building.

The building's age may give some clues as to the likely type and format of connections, but there are many variations in design approaches. Consequently it is recommended that all panel connections are reviewed carefully, regardless of building age.

It is critical that the influence of 'locked-in' stresses that exist due to the effects of aggregated earthquake deformation is considered. This should be addressed in two ways:

1. Direct observation: a brittle connection that shows signs of cracking may be at the limits of its ability to resist further loads. Conversely a ductile or flexible connection that is only moderately deformed (say less than 50% of its potential movement capacity) may be considered to have adequate capacity to resist future earthquake displacements.
2. Analysis: the building analysis should allow for the effects of deformation including consideration of the connection ductility. That is, if the connections are brittle, the building should have been analysed as a brittle building.

The damage threshold indicators in section 3.4 of this document may give further guidance, noting that such cases should be assumed to be non-ductile for this assessment.

Many tilt panel structures in Canterbury use weld plate connectors. These are often brittle, and generally have no allowance for shrinkage over the structure's length. The concrete around the connections is often damaged by expansion due to the heat build-up from welding. Where multi-bay structures contain panels over a significant length, it was common even before the earthquakes to see cracked connections at reasonably frequent intervals, as a result of shrinkage or thermal movements These 'prior failed' connections may have had an influence on the structure's behaviour as a whole, either by acting as stress relief points, focusing movement into a single location, or by reducing overall capacity. In such cases, the building's capacity with and without the weld plate connection may need to be separately considered.

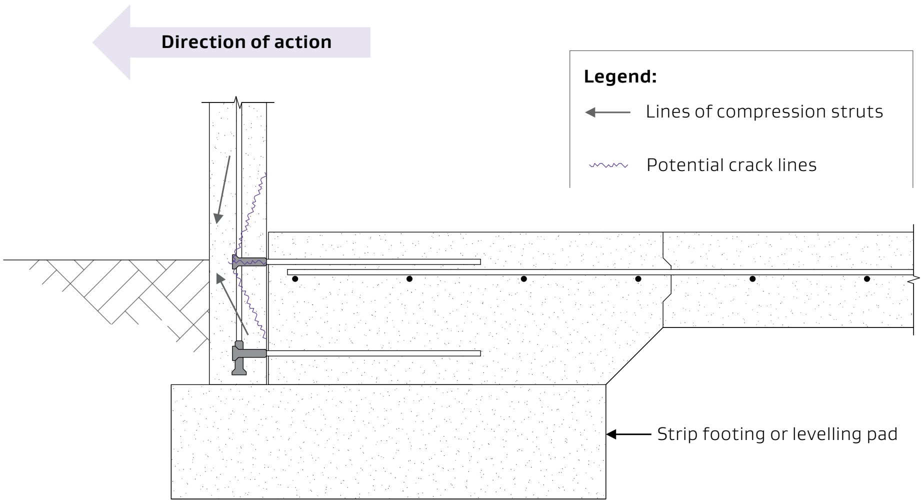
Bolted connections offer more ductility, provided that the anchorage of the connection into the panels is fully developed. Cast-in sockets or similar, with the anchor rods that are typically (but not always) included, are not generally capable of preventing a crack occurring – instead they offer a back-up to prevent a full pull-out failure. The success or otherwise of this detail depends on the reinforcement detailing within the panel around the insert, requiring adequate continuous trimmers at panel edges to prevent the connection failing completely.

Another common practice in older tilt panel buildings is to have bolted connections into proprietary socket inserts welded to anchor bars. Many of these inserts were manufactured from easy-cutting steel that had a high content of alloy materials making it unsuitable for welding. These are likely to be brittle.

The assembly's ductility should be considered carefully. Although steel is a ductile material, the way that it is detailed determines the connection ductility. For example, in a panel to floor connection, if a hooked bar is inadequately anchored, it will cause a cone pull-out, well before the bar will yield at the panel-floor interface.

The location of panel fixings with respect to likely panel cracks should also be considered. For example, a common base detail in more recent years relies on cantilever action being achieved with cast-in fixings at the base (refer to Figure 3.5). In the event of a crack developing, the anchor has its capacity instantly reduced and cannot be relied on to intercept the compression struts required to complete the load path.

Figure 3.5: Panel base fixing detail illustrating potential crack locations



In order to identify issues with connections below ground level, localised external excavation and/or partial slab removal may be required.

Consideration also needs to be given to the behaviour of panel connections in fire, according to factors such as the proximity to the boundary and the spread of fire requirements to adjacent structures.

3.3.5.2 Other panel connections

Other connections typically involve the restraint of panels for face loading and do not determine the behaviour and capacity of the primary lateral load resisting system.

All other connections should be assessed for both strength and deformation capacity. In the case of connections that have failed, it is important to consider whether the failure is simply as a result of the loads being much higher than expected, or if it is an indicator of more unsatisfactory performance or substandard construction. In the latter case, this may be an outcome of gross under-sizing of the connection, or from premature failure of other parts of the system leading to an alternative load path that over-stresses the connections.

It is therefore critical that the assessor determine which of these has happened and use this to inform the subsequent repair or replacement strategy.

3.3.6 Configuration issues

The overall configuration of any building will have a significant influence on its behaviour. Specific configuration issues are described in the following topics.

3.3.6.1 Cantilevered columns

A significant number of industrial buildings have the primary lateral load resistance supplied by cantilevered columns, with base stability being provided either from postholes, or from large pads. Where such buildings are on soft or liquefiable material, large rotations may occur at the base, imposing significant displacements and/or forces at high level. This may also result in diagonal crack patterns at end bays, where the much stiffer end wall may result in significant warping actions on the end panels, with ensuing cracking and damage. These cracks may superficially resemble settlement cracks, but this may be ruled out by completing a level survey.

Where these elements provide gravity support to roof members the connections to the roof members need to be inspected to check that the vertical support has not been compromised.

3.3.6.2 Oversize lintel panels

In many tilt panel industrial buildings, the entire perimeter wall cladding system consists of precast concrete panels, even where there are large (often multi-bay) openings in the walls. Typically this form of opening incorporates precast concrete lintel panels spanning horizontally between adjacent panels either side of the opening. This creates a stiffness incompatibility with the adjacent solid wall elements, often resulting in differential settlement being concentrated at the openings. This puts considerable further stress on the lintel panel connections at these locations, in excess of the demands resulting from shaking damage alone.

The connections to these lintel panel elements should be carefully checked, particularly where the panels are located above key egress points for the building.

3.3.6.3 End bay compatibility

Where there has been excessive movement in the portal frames adjacent to stiff end walls, there is potential for excessive damage to the cladding. Guidance is given on acceptable deflection limits in Table C1 of NZS 1170.0, relating to deflection of portals under Es or Ws (SLS earthquake and wind actions respectively) load cases, noting that this is recommended guidance only.

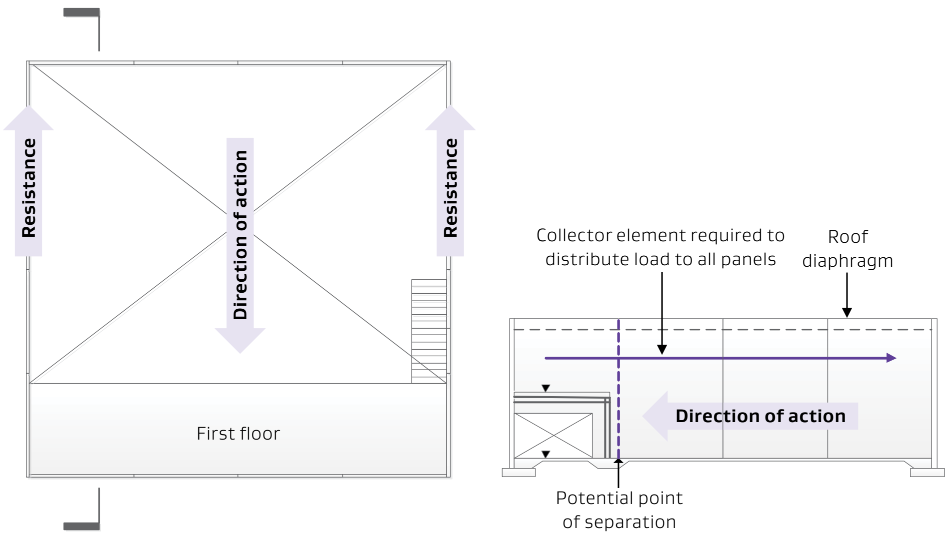
3.3.6.4 Uneven mass and lateral load resistance distribution.

Several instances were observed where poor configuration of mass and lateral load resistance was a significant factor in the poor performance outcome in the earthquake. Two cases are described below:

› **Example building 1**

In this case, the building has a partial first floor at one end, supported in the longitudinal (Y) direction by a line of panels at each side (refer to Figure 3.6). Although there was probably sufficient in-plane capacity in the panels at each side of the building to support the additional load of the first floor and end panels, there were no (or insufficient, in either stiffness or strength) collector elements to drag the load back along the entire length of the wall. In the absence of such collector elements, there was insufficient strength and/ or stiffness in the panel connections and so the front of the building separated from the remaining structure.

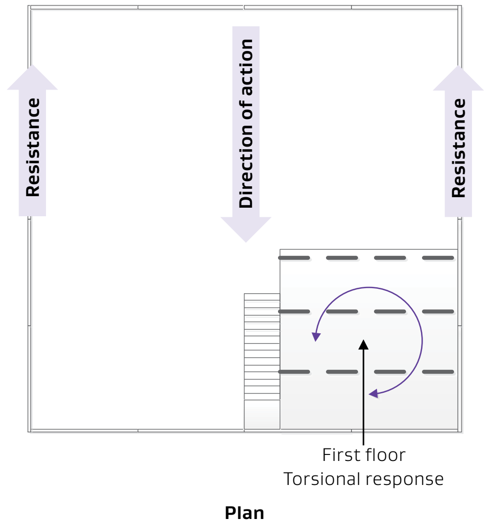
Figure 3.6 (a): Schematic plan layout of example building 1 featuring an incomplete load path



› **Example building 2**

Another typical example of this type of failure is in buildings with isolated areas of mezzanine floor, typically in corners of buildings, where one or two sides of the mezzanine have their lateral load resistance provided by open structural steel frames. Although the frames (shown as dotted lines) may have been designed on a tributary width basis, on the assumption of floor diaphragm flexibility, the stiffness incompatibility may result in additional seismic loads being applied to the panels, with resulting overstressing of panels or (more likely) the connections.

Figure 3.6 (b): Schematic plan layout of example building 2 incorporating a lack of symmetry



3.3.7 Slab on grade performance

The ground floors of most industrial buildings are concrete slabs on grade.

> **Note:**
>
> This section is not applicable to industrial buildings with suspended ground floors.

Slabs on grade are susceptible to the effects of liquefaction and differential settlement. In many cases liquefaction may have occurred at depth but has not resulted in surface expression through ejecta. In such cases, there may still be severe differential settlement at ground level. Differential settlement can also result from consolidation of soft soils without liquefaction, particularly under the action of vibration from specialised plant and equipment or heavy foundation loads. This is generally of less concern under industrial buildings, which typically do not have heavy foundation loads. Such effects should be ruled out of consideration before coming to any conclusion regarding settlement.

In assessing slabs on grade it is critical to ascertain the structural function (if any) of the slab. In some cases, the slab on grade may be required to act as a tie element across the building, or may be required to assist in providing out-of-plane stability of cantilevering walls under fire or post-fire actions.

The form of reinforcement in the slab should be determined. There are four main categories of slab reinforcement:

1. Unreinforced
2. Cold-drawn mesh reinforced
3. Ductile mild steel reinforced
4. Post-tensioned (with high-tensile steel)

Assessment of the impacts of slab movement is highly dependent on building use.
Refer to section 3.4 of this document.

In areas where liquefaction has been identified, sub-floor investigation to detect voids under the slab may be appropriate to ensure uniform support is still provided for the floor. This could be by falling weight deflectometer (FWD), or ground penetrating radar (GPR).

Where possible, it is important to consider the significance of prior movement (construction tolerance and historical settlement). If it is assumed that all variation is a consequence of the earthquake sequence, the impact of the earthquake movement on the structure may be over-estimated.

3.3.8 Fire considerations

The extent to which fire needs to be considered depends on the repair or rebuild approach. This is addressed with respect to the applicable sections of the Building Act in sections 4 and 5 which follow.

It is important in the assessment phase to note information that may be required in the subsequent phases. This includes:

- where the title boundaries may require specific fire separations to be repaired or maintained

  > **Note:**
  >
  > in buildings that cross title boundaries, this may impact on internal structure as well as exterior walls.

- the location of fire egress paths that need to be considered with respect to maintenance of the protection systems
- where elements of vulnerable structure may adjoin egress paths or mustering points.

3.3.9 Inter-tenancy (party) walls

Inter-tenancy walls are generally performing more than one function. In addition to providing seismic load resistance, they will almost certainly have to satisfy fire (and possibly acoustic) separation requirements. Care is needed to establish if fire performance is dependent on lateral support provided by elements each side of the adjoining structure. This will apply to all walls on title boundaries even if the structure is continuous over more than one title, unless there are legal covenants in place that allow fire-rating requirements to be waived. This should be verified with the owner.

3.3.10 Underground tanks and basements

Many underground tanks and basements have floated as a consequence of liquefaction. Where these structures are under buildings or parts of buildings, this can have a significant influence on the structure above. In particular, where the underground structure is not under the entire building's footprint, increased differential settlement has often resulted. While basements are generally easy to detect, underground tanks are not always obvious. Assessors are advised to ensure that they have checked with owners for the presence of these structures before commencing work on site.

> **Note:**
>
> Many underground tanks have been simply abandoned over the years, rather than fully removed. If unusual ground movement is observed, the presence of abandoned tanks should be considered. This may mean that there is ground contamination, which may require special treatment if soil is to be removed from the site.