PART C

Reinforced Concrete Masonry Buildings

C11

Non-EPB Seismic Assessment Guidelines

ONLY TO BE USED FOR SEISMIC ASSESSMENTS OUTSIDE THE EPB METHODOLOGY

2025

Foreword

The Joint Committee for Seismic Assessment and Retrofit of Existing Buildings (JC-Sar) is responsible for the joint oversight of the system used to assess, communicate, manage and mitigate seismic risk in existing buildings. It reviews how the guidelines are functioning in practice, identifies areas that require further input and development, and either advises on or assists in the development of proposals for work programmes that contribute towards these objectives. The Joint Committee includes representatives from The Natural Hazards Commission Toka Tū Ake, the Ministry of Business, Innovation & Employment, and the technical societies (NZGS, NZSEE, SESOC).

The Joint Committee's Vision is that:

- Seismic retrofits are being undertaken when necessary to reduce our seismic risk over time while limiting unnecessary disruption, demolitions and carbon impacts, promoting continued use or re-use of buildings.
- Decisions on retrofitting are informed by an appropriate understanding of seismic risk and are aligned with longer term asset planning.
- Seismic assessment and retrofit guidelines help engineers focus on the most critical vulnerabilities in a building, serve the needs of the market and regulation, and evolve
- through a stable ongoing cycle allowing new knowledge and improvements to be included in a predictable manner, including the consideration of objectives beyond life safety.
- Engineers are supported in the implementation of Seismic Assessment and Retrofit Guidelines through a range of training and information sharing strategies, including tools for risk communication to manage unnecessary vacating of buildings.
- Society is informed about the level of risk posed by existing buildings.

Acknowledgements

Updates encompassed in this document were prepared by a project team under the direction of the Joint Committee (JC-Sar). The project team comprised contributing authors plus a technical review group.

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This document is managed by the Joint Committee for Seismic Assessment and Retrofit of Existing Buildings. It may be downloaded from design.resilience.nz.

Refer to the following pages for a summary of the key changes from previous versions.

Please visit design.resilience.nz to provide feedback or to request further information about these Guidelines.

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This section is part of the Non-EPB (Earthquake-Prone Building) Seismic Assessment Guidelines which constitute a proposed technical revision to the July 2017 EPB Seismic Assessment Guidelines. The Non-EPB Seismic Assessment Guidelines may be used for general commercial Detailed Seismic

Assessments for non-EPB purposes. It is to be used in conjunction with Part A of the EPB Seismic Assessment Guidelines.

Engineers engaged to assess buildings identified by a territorial authority as being potentially earthquake prone in accordance with the EPB Methodology must continue to use EPB Seismic Assessment Guidelines (1 July 2017) as these are referenced in the Methodology.

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C11. Reinforced Concrete Masonry

C11.1 General

C11.1.1 Scope and outline of this section

This section provides guidelines for performing a Detailed Seismic Assessment (DSA) for existing Reinforced Concrete Masonry (RCM) buildings and components. It covers the recent history in New Zealand, known seismic weaknesses and earthquake performance of RCM. It also provides information to enable the assessment of RCM sections, members, components and sub-assemblies in buildings. Few buildings are exclusively RCM, so it is expected that this section will be used in conjunction with other sections. Unreinforced concrete masonry structures are not addressed, instead refer Section C8 (Unreinforced Masonry Buildings). Similarly, RCM walls used as infill in moment resisting frames should be assessed using Section C7, which deals with the specifics of this combination of structural elements.

This section is aimed primarily at low rise RCM buildings, due to their prevalence in New Zealand. Engineers assessing high rise RCM buildings may use this section but also should consider if further assessment of the critical elements is needed. This section is intended to leverage the knowledge in Section C5 (Concrete Buildings), and reference is made to Section C5, where appropriate.

The overall aim is to provide engineers with:

- an understanding of the underlying issues associated with the seismic response of RCM buildings (including the presence of inherent vulnerabilities or weaknesses), and
- supplementary information for RCM to allow the application of assessment tools in Section C5 and other sources.

This section covers in turn:

- typical building practices and observed behaviour of RCM components in earthquakes (refer to Sections C11.2 to C11.3)
- structural deficiencies, material properties and testing (Sections C11.4)
- specific considerations for assessing RCM components using the tools in Section C5 (Sections C11.5 to C11.6)
- global building capacity considerations (Section C11.7), and
- alternative assessment techniques (Section C11.8)
- methods to improve RCM components (Section C11.9).

Appendix C11A summarises material testing options for RCM components.

> **Note:**
>
> The effects of Soil-Structure Interaction (SSI) in terms of seismic performance, modifications of demand and development of mixed mechanisms are discussed in Section C4.

C11.1.2 Definitions and acronyms

Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable yield capacity is exceeded. A brittle structure has little ability to deform before it fractures.
Concrete block	Hollow concrete masonry blocks generally complying with NZS 4210:2001 or its preceding or superseding standards. Other forms of masonry unit (such as stone or clay bricks) are not covered by Section C11.
Detailed Seismic Assessment (DSA)	A seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor, ceiling, or braced roof structure) that is connected to the vertical elements around it and that distributes earthquake lateral forces to vertical elements, such as walls, of the primary lateral system. Diaphragms can be classified as flexible or rigid.
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Elastic analysis	Structural analysis technique that relies on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material nonlinearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis, modal response spectrum analysis, and elastic time history analysis.
Nonlinear analysis	Structural analysis technique that incorporates the material nonlinearity (strength, stiffness and hysteretic behaviour) as part of the analysis. Includes nonlinear static (pushover) analysis and nonlinear time history dynamic analysis.
Non-Specific Engineering Design (non-SED)	RCM designed following the requirements of NZS 4229:2013, or its preceding or superseding standards
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also function as the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects, this is determined using probable material strengths. For geotechnical issues, the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Reinforced Concrete Masonry (RCM)	RCM describes construction using all the following components: • Hollow concrete masonry blocks generally complying with NZS 4210:2001 or its preceding or superseding standards. Other forms of masonry unit (such as stone or clay bricks) are not covered by Section C11, even if these units are reinforced. • Mortar used to connect adjacent concrete blocks prior to grouting. • Reinforcing steel within the concrete block, or in the grout between concrete blocks. • Grout filling the internals of all, or a subset of the concrete blocks (partial grouting).
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items.

Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Specific Engineering Design (SED)	RCM designed following the requirements of NZS 4230:2004, or its preceding or superseding standards
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100%*NBS*. Note that an aspect of the building structure scoring less than 100%*NBS* but greater than or equal to 67%*NBS* is still considered to be a SW even though it is considered to represent an acceptable risk.

C11.1.3 Notation, symbols and abbreviations

Symbol	Meaning
$\%NBS$	Percentage of new building standard as calculated by application of these guidelines
d	Distance from extreme compression fibre to centroid of tension reinforcement (mm)
d'	Reduced distance from extreme compression fibre to centroid of tension reinforcement, accounting for lack of hooked horizontal reinforcement (mm)
E_{m}	Young's Modulus (MPa) for RCM
f_{cb}	Characteristic concrete block compressive strength (MPa)
f_{g}	Characteristic compressive strength of grout (MPa)
f_{mortar}	Characteristic compressive strength of mortar (MPa)
f'_{c}	Probable concrete compressive strength, as defined in Section C5 (MPa)
f'_{m}	Probable concrete masonry compressive strength (MPa)
L_{d}	Theoretical development length, refer Table C11.5 (mm)
L_{dh}	Theoretical development length of a hooked bar (mm)
V_{bm}	Basic shear strength of masonry, as defined in Table C11.6 (MPa)
V_{g}	Maximum permitted masonry shear strength, as defined in Table C11.6 (MPa)
α	The ratio of the net concrete block area to the gross area of the concrete block
$\varepsilon_{\mathrm{cm,max}}$	Maximum concrete masonry compressive strain
ϕ	Strength reduction factor
γ_{age}	Strength gain factor due to aging beyond 28 days
γ_{prob}	The ratio of probable strength to characteristic strength for reinforced concrete masonry
μ	Element displacement ductility
μ_{f}	Coefficient of friction

C11.1.4 Assessment approach

Section C2 outlines the general assessment approach of these guidelines, however the frequent usage of RCM for squat, low rise construction can also enable a faster assessment approach in certain circumstances. The assessing engineer is encouraged to consider whether crude conservative assumptions will enable faster assessment while still potentially resulting in scores exceeding 100%*NBS*. For example, a long wall may be able to be shown to carry its shear and out-of-plane loading using the minimum provided f'_m. This approach depends on the specifics of the building and will not always be appropriate or efficient.

Conversely, when assessing atypical construction or when the lateral system relies on only a few elements and/or higher levels of ductility, it is expected that greater care will be exercised. For example, mid-rise buildings designed with flexural RCM walls need careful consideration of any plastic hinge development, and the potential for boundary element buckling. In such circumstances, Section C5 is recommended as a further source of guidance. In all instances, the level of assessment is expected to reflect the consequences of failure of the assessed mechanism.

C11.2 Reinforced Concrete Masonry Construction Practices in New Zealand

C11.2.1 General

Hollow concrete blocks used without reinforcing are known to have been produced in Wellington as early as 1904, with more producers opening from Whangarei to Invercargill by 1910 (Isaacs, 2015). The hollows in these blocks were primarily used to reduce weight, rather than provide space for internal grout. The concrete company, Firth, claims the first ‘machine made’ concrete blocks were manufactured by an earlier Firth entity in 1938, indicating an ability to mass produce blocks.

The early use of grouting and reinforcing in concrete blocks is less well understood, but inference may be made from the history of New Zealand standards.

From the first New Zealand masonry code in 1948, RCM has been separated into two categories; elements needing Specific Engineering Design (SED), and elements that do not (non-SED). In 1985-1986, new standards split SED and non-SED further by placing them into separate documents. Non-SED was developed to permit construction of RCM structures without engineering input and allowed minimal site supervision during construction. The history of SED and non-SED standards is summarised in Section C11.2.2.

Note:

It is convenient when reporting the history of RCM to focus on SED and non-SED, and this is how it is presented in Section C11.2.2, but other factors may be just as determinative on the structural performance. A well-constructed non-SED wall may well perform better than a poorly executed SED wall (and this has sometimes been observed after intrusive inspections of both SED and non-SED walls). This reality is reflected in the default material properties provided for RCM in this guidance, which rarely relate to whether the structure is SED or not.

C11.2.2 RCM standards

RCM materials, loading and detailing requirements have evolved with changing RCM standards (Figure C11.1Figure C11.1). An overview of the key history of these standards is provided below. Concrete block material standards are not explicitly covered here, but a summary is tabulated in Section C11.4.4.2.

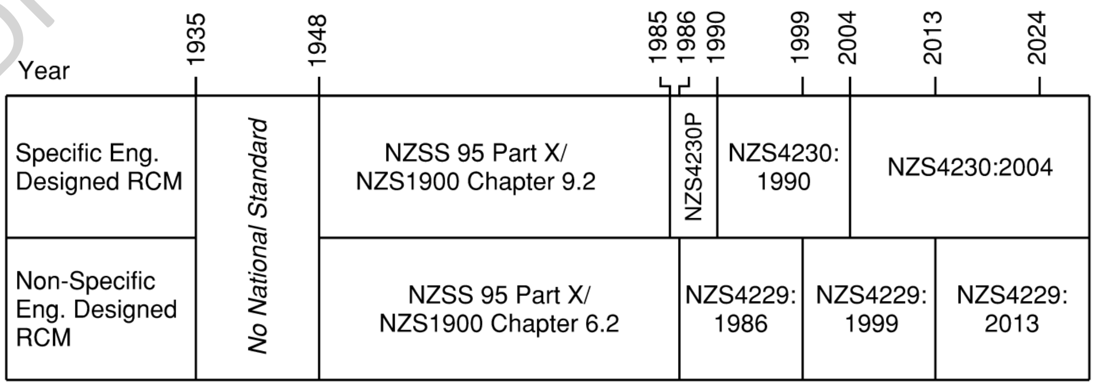


Figure C11.1: History of Reinforced Concrete Masonry standards in New Zealand (adapted from Smith and Devine, 2011)

Note:

The ease of constructing RCM, and the comparative lack of enforcement of early RCM regulation, means that the timeline presented here should be considered an informative guide only. It is expected that early compliance with the listed standards will vary significantly between buildings. Progressively greater compliance was developed in later years.

C11.2.3 NZSS 95 Part X/NZS 1900 (both SED and non-SED)

The first national New Zealand masonry standard was published in 1948 under the title "Masonry Construction" in Part X of NZSS 95 (Smith and Devine, 2011). The standard included specification of masonry material types and quality. Construction and workmanship conditions requirements were also provided, with differing allowable stresses for differing levels of supervision. The standard covered clay units, concrete blocks and concrete bricks, and included both reinforced and unreinforced masonry.

While NZSS 95 was intended as a national standard, it needed formal adoption by municipal and county councils. This process was gradual, and some councils adopted only parts of the standard. By 1963, NZSS 95 was adopted by 'most' municipal councils, but only half of county councils (Isaacs, 2022).

NZSS 95 Part X was reformatted and republished as NZS 1900 in July 1964. While no new material was added, masonry requirements for 'buildings not requiring specific design' were separated into Chapter 6.2. Specific design of masonry was placed in Chapter 9.2. Again, these standards required adoption by councils, and this adoption process was gradual (Davenport, 2004).

C11.2.4 NZS 4230P:1985 (SED)

NZS 4230P was cited in 1985 and brought limit state design and ductility concepts to the masonry standard. It was structured to follow a similar form to the concrete standard NZS 3101:1982, to allow reference to the concrete standard for elements of masonry design (Priestley, 1985).

Three grades of masonry strength were introduced, depending on the extent of construction observation by the engineer; Grades A, B and C, (permitting f'_m of 8 MPa or greater with testing, 8 MPa and 4 MPa, respectively). The following grade descriptions were provided by Priestley in his commentary on the then new code (Priestley, 1985):

Grade C masonry, the minimum standard, is unsupervised and is intended only for structures not requiring specific design, or in special circumstances where inspection is impractical.

Grade B masonry, the standard grade, will be designed and inspected by an engineer experienced in this form of construction. Inspection of reinforcement placing, and grouting, is emphasised. End zones (potential plastic hinge regions) of Grade B masonry must be all-cells filled.

Grade A masonry, recognises that reliability and structural performance are improved when all cells are completely filled and work is closely supervised at all critical stages. More *stringent inspection is required than for Grade B masonry to ensure quality construction throughout.*

The masonry design grade was required to be identified on the construction drawings, although assessing engineers have frequently reported that this information is missing.

C11.2.5 NZS 4229:1986 (non-SED)

NZS 4229:1986 was intended to enable some RCM design using simplified rules for people with limited engineering training (Wylie, 1993). Its usage was limited to two storeys, or three storeys where the third was constructed of timber. Suspended floors and roofs were limited to lightweight timber construction. Plan size limits were also specified. Partially filled masonry was permitted in lower seismicity zones, within limitations. Parapets were permitted up to 1 m in height and could be partially filled. Non-SED retaining wall design was also covered by the standard.

C11.2.6 NZS 4230:1990 (SED)

The 1990 revision made limited changes to the SED standard, including increased shear capacity, and a revised section on veneers. Development lengths were also adjusted to account for the increasing capacity of ‘high yield’ bars from 380 MPa to 415 MPa (Gaerty, 1991).

C11.2.7 NZS 4229:1999 (non-SED)

This non-SED revision updated the standard to comply with NZS 4203:1992 and updated design tables in accordance with research findings. Partial fill was permitted in all seismicity regions. Non-SED RCM was also permitted for buildings with concrete diaphragms (NZS 4229:1999).

C11.2.8 NZS 4230:2004 (SED)

The 2004 revision of NZS 4230 included raising the permitted f'_m for Grade B masonry to 12 MPa, not because changes were made to the material and construction requirements, but because additional data was available to support a less conservative value. The standard was reformatted and made compatible with NZS 1170 loading standards (Cathie, 2003).

C11.2.9 NZS 4229:2013 (non-SED)

In 2013, the non-SED standard was updated to reflect the NZS 1170.5 loading standard and provided designs varying by soil class (soil classes A-E). Cantilever and retaining walls were also revised for the updated seismic loadings (NZCMA, 2012).

C11.2.10 Construction practises

Concrete masonry block construction can be categorised into fully filled, partially filled and completely unfilled construction. Unfilled construction is covered by Section C8 (or Section C7 when it is infill in a moment frame) and is not discussed further here. Partially filled blockwork comprises horizontal and vertical reinforcement in some, but not all, block cells. The layout and spacing of this reinforcement should generally meet the minimum requirements of NZS 4229:2013 – for example a block wall with a single reinforced and grouted cell should not be considered ‘partially filled’. All cells with reinforcement are required to be filled with grout. Between these locations, only URM concrete block and mortar is present (refer to Figure C11.2). Fully filled block may not have reinforcement in every cell, however all cells are grout filled.

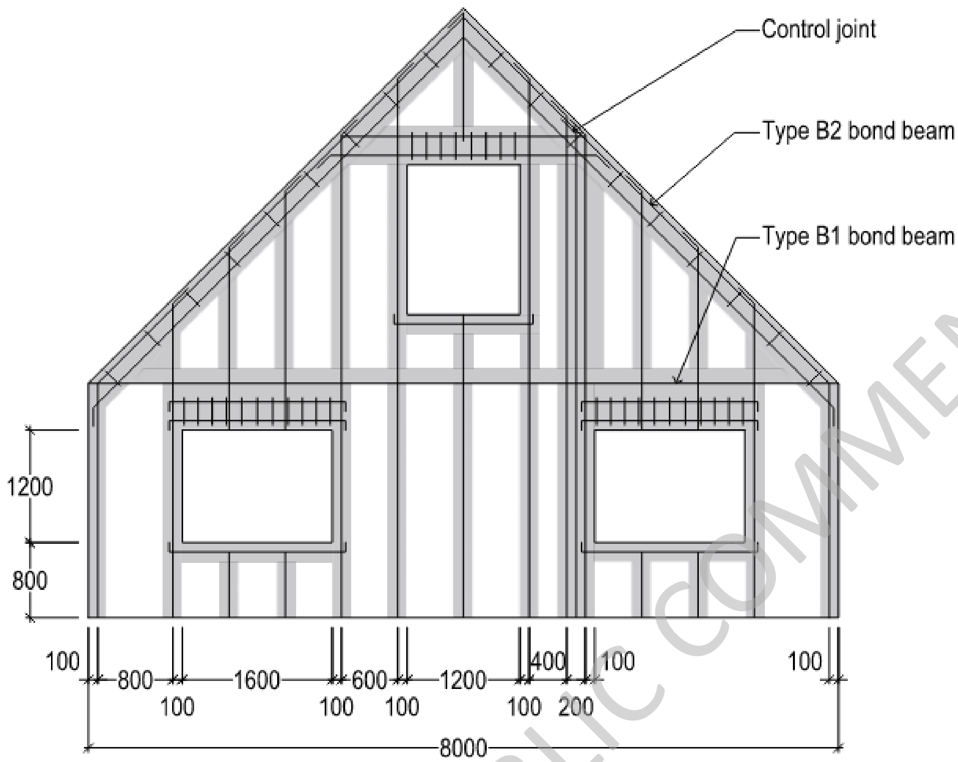


Figure C11.2: Example of filled grout cells in a partially filled RCM wall. Light grey areas denote filled cells (adapted from NZCMA, 2012)

RCM veneer walls are an occasional feature in both residential and non-residential RCM construction (Fikri et al., 2019). Veneers comprise an outer skin made of clay or concrete masonry bricks, which is laterally connected via ties to an interior skin made of RCM. The walls are separated by a cavity which is commonly in the order of 40-100 mm. Ties often degrade/ corrode over time and should be inspected before they are relied upon for assessment.

Veneer walls may be indicated by weep holes or air vents at the bottom of walls, although weepholes are sometimes also provided in partially filled single skin walls. Weep holes may appear in regular locations as vertical slits where mortar would usually be present in the bottom masonry course.

C11.3 Observed Behaviour of RCM Components

C11.3.1 General

Many of the RC damage observations discussed in Section C5 are also relevant to RCM, but some additional information is included below. Common damage characteristics include:

- face shell spalling of the concrete block and diagonal cracking due to in-plane shear demands
- face shell spalling of the concrete block and grout crushing in boundary elements (or wall ends) due to in-plane flexure
- yielding of reinforcement and destruction of concrete block shell faces due to out-of-plane flexure
- increased damage at connections between RCM spandrels and walls, and other areas with stress concentrations, and
- debonding of reinforcing bars at lap locations, areas of low reinforcement cover or plastic hinge zones.

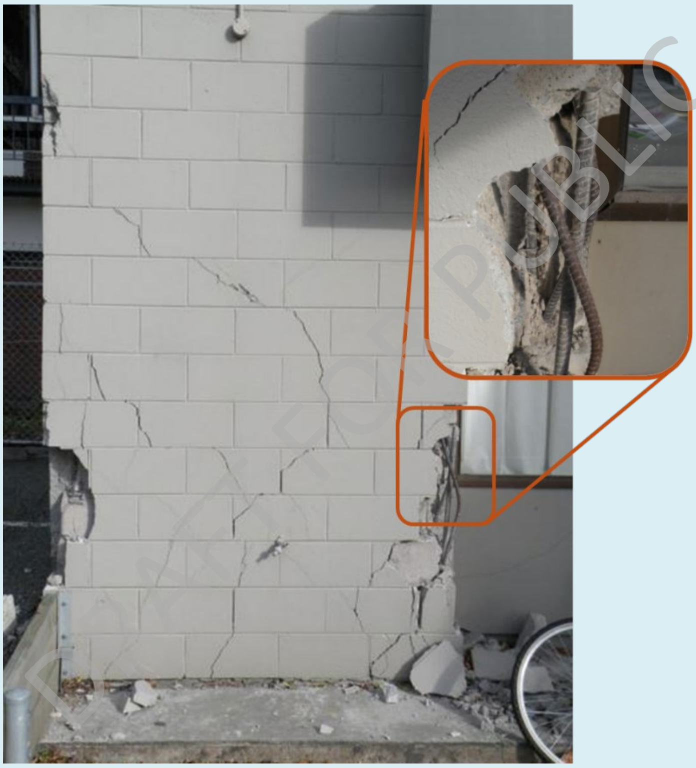
As RCM generally does not have confining steel, exceedance of capacity is often characterised by spalling of the concrete block shell facings, then loss of sections of unconfined grout and buckling of the reinforcement.

Examples of earthquake damaged RCM are presented in Table C11.1. The images have been selected to show a variety of damage types and severity of damage witnessed in the 2010/2011 Canterbury earthquake sequence.

Note:

No fatalities were directly attributed to RCM in the 2010/2011 Canterbury earthquake sequence. As a result, RCM construction was not a focus for the subsequent Canterbury Earthquakes Royal Commission of Inquiry. It can be said that the presence of reinforcement and the generally low aspect ratios of RCM components have historically resulted in better seismic performance than URM and reinforced concrete. However, as illustrated below, RCM remains susceptible to earthquake damage.

Table C11.1: Examples of RCM damage observed in the 2010/2011 Canterbury earthquake Sequence

Recorded Damage	Observations
	This wall was partially filled, and displays a 'yield line' crack which intersects the corner of a second parallel wall (see arrow). Photo credit: Greg Cole
	A flexural wall that has spalled sections of concrete block and grout, and buckled bars at the wall boundaries. The highlighted section shows a bar lap which has debonded. Note the lack of hooked horizontal rebar which could have increased the ductility capacity of this section. Some shear cracking is observed, but it has not developed to the extent that the gravity load path is compromised. Photo credit: Waipapa Taumata Rau University of Auckland

Recorded Damage	Observations
	Toe crushing of return wall and vertical bar buckling at wall boundary due to lack of horizontal restraint. Photo credit: Waipapa Taumata Rau University of Auckland
	Two views of the same column showing extensive concrete block and grout loss. Bar laps are again present where the reinforcement has debonded. Photo credit: Waipapa Taumata Rau University of Auckland
	Concrete block face shell spalling due to insufficient reinforcement cover. Photo credit: Waipapa Taumata Rau University of Auckland

Recorded Damage	Observations
	Damage to an RCM wall supporting an exterior concrete masonry veneer. Significant loss of concrete, block and grout, with debonding of reinforcement and longitudinal bar buckling. Photo credit: Waipapa Taumata Rau University of Auckland
	Concrete block and grout loss attributed to horizontal reinforcement lap splice failure. Photo credit: Waipapa Taumata Rau University of Auckland

Recorded Damage	Observations
	Sliding shear failure, spalling of concrete block, grout and loss of bond to reinforcement. Photo credit: Waipapa Taumata Rau University of Auckland
	Wall shear failure showing spalling of concrete block shell faces. Note insufficient cover to horizontal reinforcement. Photo credit: Waipapa Taumata Rau University of Auckland
	Spalling and concentration of damage at the interface of spandrel and wall units. Photo credit: Waipapa Taumata Rau University of Auckland

Note:

Section 6.5 of FEMA 306 is a further useful resource that contains clear illustrations of common RCM failure mechanisms

C11.4 Material Properties and Testing

C11.4.1 General

For RCM components, key material-related data for the assessment include:

- concrete block and grout strength
- steel yield strength, probable tensile strength, probable strain capacity and the expected variation in its properties.

Mortar strength is not typically critical for RCM, as the compressive capacity is mostly governed by the grout and concrete block strength (Paulay and Priestley, 1992).

Information on the mechanical properties of the RCM component materials, and the intended construction supervision can be sourced from:

- the construction drawings
- the original design specifications
- original test reports
- knowledge of the practices of the time
- site observations of quality, and/or
- in-situ testing.

In the absence of specific information, default values for the mechanical properties of the reinforcing steel and concrete masonry may be assumed in accordance with the relevant standards and practices at the time of construction, after first making an assessment on general material quality via site inspection. The following sections provide the intended default values.

RCM requires skilled construction and identifying defects can be more difficult than for reinforced concrete construction. The structural consequence of poor construction can be high. This can be seen in NZS 4230:2004 where supervised masonry is designed assuming $f_m' = 12$ MPa, whereas unsupervised masonry is restricted to $f_m' = 4$ MPa – a reduction of 2/3rds. Unless adopting the lower bound capacity given in Section C11.4.4.2, site investigation including consideration of common construction defects should form part of the DSA assessment.

C11.4.2 Site investigation

Poor construction of RCM is often not obvious by visual inspection, due to the grout being contained within the concrete block. Common construction defects include:

- poor compaction of grout, leading to voids (Figure C11.3Figure C11.3) and potential corrosion of reinforcement
- lapping of vertical starter bars in plastic hinge zones with effectively no confinement
- movement of the reinforcement during grout filling, potentially resulting in lower out-of-plane flexural strength than intended in the design, and
- insufficient clearing out of the RCM wall base interior prior to grout filling, causing poor bond between the base of the wall and its supporting structure.

Note:

Assessing engineers should be particularly wary of unintended grout voids within RCM. Grout voids are common, frequently overlooked, and can significantly affect the strength and ductility of the RCM element.

These guidelines address the uncertainty of this potential defect by significantly reducing RCM strength if no site testing for grout voids is undertaken. The recent development of multi-scanning tools permits non-destructive and comparatively cheap void inspections. The alternative drilling method for grout voids is destructive, but is cheap and can be readily made good. Checking for grout voids is considered an essential step in RCM strength assessment, unless the minimum f'_m specified in Section C11.4.4.2 is adopted.

Figure C11.3: Example of partial wall failure due to voids in 'filled' cells. Photo credit: Waipapa Taumata Rau University of Auckland.

Grout voids are not limited to non-SED construction. Figure C11.4 shows photos taken at a 2024 construction site, with standard SED site monitoring. The presence of voids was found by happenstance, which resulted in further investigation works. Ground Penetrating Radar was used to scan for voids, and multiple areas were subject to repair works.

Figure C11.4: Top and Left: Voids discovered in a SED RCM wall, built in 2024. Right: Ground Penetrating Radar scan showing areas of low density. Note only some purple zones are voids, a skilled scanner operator was needed to identify the actual voids in these scans. Photo credit: Beca Ltd

Grout voids are more likely when:

- smaller block widths are used, such as 15 series (140mm wide) block
- high reinforcement ratios are used
- horizontal reinforcement is lapped side by side instead of one bar on top of the other
- grout pour methods are used that are not the high lift grouting method, or
- expansive admixture is not added to the grout.

These factors should be considered when selecting areas to test for grout voids. Testing should also consider whether partially filled block is likely at the site (which can be tested by checking adjacent cells for grout).

The extent of effort for the site inspections should be determined by the significance of the structural element, however at minimum site inspection involves all the following:

- Scanning to confirm presence of reinforcement.
- Void scanning or drilling with a ~8 mm diameter masonry bit 50-80 mm into the masonry to confirm whether grout is present in at least three separate locations. Drilling

into the mortar (rather than the block) may enable easier repair, although some testers have reported block drilling to be easier.

- Void scanning or using a hammer to knock on a representative sample of RCM to determine areas of void and area of fill. The knocking method requires experience to interpret whether a knock sounds ‘hollow’ and may need calibration against known areas (such as the drilling locations).

Testing of masonry anchors should also be considered when undertaking site inspections. Masonry anchors are discussed further in Section C11.4.8.

Drilling is an intrusive investigation method and can be substituted with use of the non-destructive alternatives. Options include: Ground Penetrating Radar (GPR), thermal imaging and stress wave propagation techniques. Note these techniques may require an experienced operator to interpret results and may need intrusive works to confirm findings. Further information about testing options is included in Appendix C11A.

Note:

Many of the tools that are used for site inspection (such as hammer drills, masonry bits, ferro scanners and GPR scanners) are available from hardware stores at prices that will be viable for some engineering firms to purchase. Note destructive testing should be properly made good.

An example of a wall scanner output is shown in Figure C11.5. Different models of wall scanner can scan different wall thicknesses, so check carefully the maximum depth of field of the scanner prior to use.

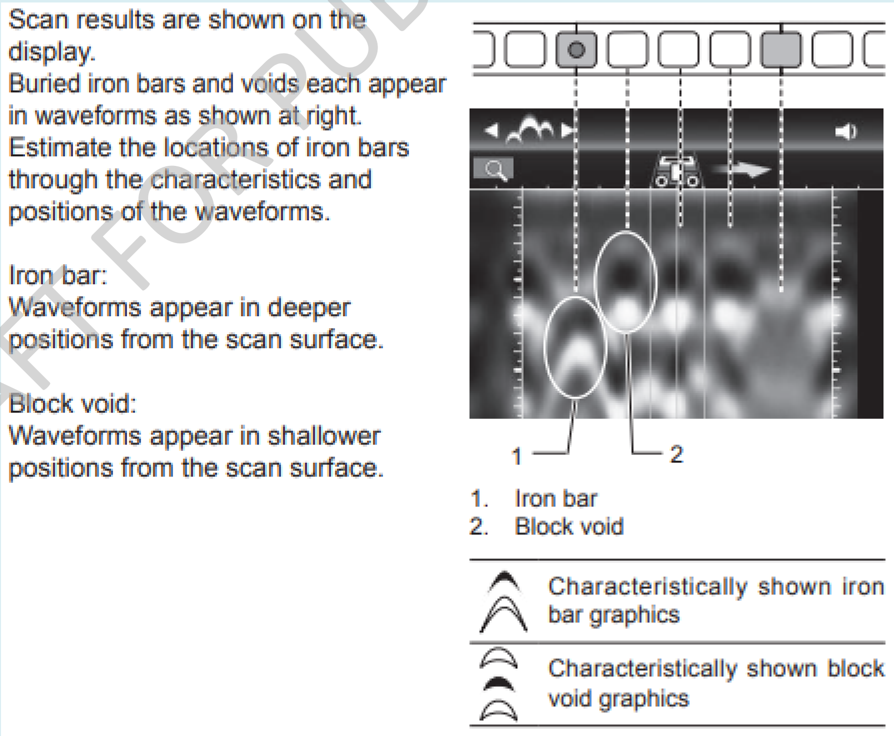


Figure C11.5: Example of wall scanner output available via hardware stores. Image source: Makita DWD181 user manual.

Note:

RCM detailing can vary widely (refer to Figure C11.6). Assessing engineers have previously found:

- discontinuous bond beams
- trimming reinforcement stopping slightly past openings, but not connecting into bond beams
- reinforcement in mortar joints but not in concrete block cells
- 100 mm high blocks made to appear like bricks, but reinforced
- concrete block masonry without reinforcement
- separated wythes of 100 mm concrete block, with bridging bond beams
- concrete blocks that contain trace amounts of ferrous material, which trigger ferrous detectors, even though reinforcement is not present
- concrete blocks filled using site-mixed concrete delivered in buckets due to lack of access to concrete pumps

Such details highlight the need for a 'sceptical' approach to site investigations, where assumptions are minimised through careful selection of intrusive inspection locations.

Figure C11.6: Top: 100 mm high concrete blocks made to look like URM. Left: reinforcement in vertical mortar beds. Right: two wythes of 10 series block separated by a cavity with bridging bond beams. Photo credit: Kent Huxford/Lewis Bradford Consulting Engineers

Note:

The extent of any in-situ material testing must be based on a careful assessment of the tangible benefits that will be obtained. It will never be practical to test all materials in all locations. In-situ testing may be justifiable in situations where the critical mechanism is highly reliant on material strengths, or perhaps relative material strengths (e.g. steel grade in interconnected beams and columns) but only when judgement based on an assumed range of possible material strengths cannot indicate an appropriate outcome. "Spot" testing to ascertain the material types in generic locations might be appropriate, but it is not intended that it be necessary to determine the range of properties present for a particular material.

C11.4.3 Strength reduction factors

When undertaking RCM assessment, the following strength reduction factors should be used:

- When calculating flexural capacity, $\phi = 1.0$.
- When calculating shear capacity against overstrength actions, $\phi = 1.0$.
- When calculating shear capacity when part of a yielding mechanism hierarchy check, $\phi = 1.0$.
- When calculating shear capacity, without consideration of yielding mechanism, $\phi = 0.85$.

Where 'consideration of a yielding mechanism' requires an evaluation of the lateral system, by comparing the ratio of demand vs capacity individually for both shear and flexure of each element in the system, $\phi = 1.0$ is still appropriate even if shear capacity is found to be critical, provided this mechanism check has been performed.

Note:

A mechanism check is strongly recommended in Section C2, so direct checks of shear capacity without consideration of mechanism is not expected in RCM assessment.

Note:

Use of probable and overstrength member and element capacities, as outlined in these guidelines, is considered to provide the required level of confidence that a mechanism will be able to develop with the required hierarchy if the material strengths can be reasonably ascertained. This means it is not intended that the engineer applies any additional factors to account for natural variation in material strengths when assessing the hierarchy within a particular mechanism.

C11.4.4 Concrete masonry

C11.4.4.1 General

This section provides default probable material properties for the assessment of reinforced concrete masonry. These values can be used for assessment of RCM structural elements in the absence of a comprehensive material testing programme.

C11.4.4.2 Probable compressive strength of concrete masonry

In the absence of specific information, the probable masonry compressive strength, f'_{m}, may be taken as calculated below.

RCM that has not been subject to intrusive investigation as described in Section C11.4.2 may be assessed with f'_m = 5.5 MPa. Higher values may be adopted after site investigation and consideration of the issues described in C11.4.2.

> **Note:**
>
> The value of 5.5 MPa was determined using NZS 4230:2004 design capacity of 4 MPa for non-SED RCM, then amplified by aging beyond 28 days and scaled to probable strength from characteristic strength.

Following intrusive investigation the probable filled concrete masonry compressive strength can be calculated as:

$$f'_{\mathrm{m}} = \left[0.6\alpha f_{\mathrm{cb}} + 0.9(1-\alpha) f_{\mathrm{g}} \gamma_{\mathrm{age}}\right] \gamma_{\mathrm{prob}} \quad \text{...C11.1}$$

where:

α	=	the ratio of the net concrete block area to the gross area of the concrete block. May be taken as 0.45
f_{cb}	=	the characteristic strength of the concrete block
f_{g}	=	the characteristic strength of the grout
γ_{age}	=	the strength gain due to aging beyond 28 days. May be taken as 1.2
γ_{prob}	=	the ratio of probable strength to characteristic strength for reinforced masonry. May be taken as 1.2.

> **Note:**
>
> This formula is an adaptation of Eqn. B-1 of NZS 4230:2004. The strength of the mortar does not feature in the compressive strength of the filled masonry, which is consistent with Paulay and Priestley (1992) and NZS 4230:2004.
>
> There is little testing of existing RCM elements to determine their insitu strength available in current literature. The above equation is considered a conservative approach that is appropriate given this absence of information. Significant increases in strength may be justified by testing the materials present on site, however testing of concrete masonry usually requires large samples (approximately 600 mm high) to capture the interaction of the grout, mortar and concrete block. This is likely to be deemed impractical for many seismic assessments.

Default concrete block, masonry strengths and mortars are summarised below, based on the material standards of the day.

Table C11.2: Default assumed concrete block compressive strength

Period	Standard	Concrete block compressive strength f_{cb}
Pre 1959		Unknown, 4.8 MPa may be adopted
1959-1985	NZSS 595 cited in NZSS 95 Chapter 9.2 Amendment No. 1 1959	Class A (External members without protective coating) – 6.9 MPa Class B (External members with protective coating. Internal walls or backings) – 4.8 MPa Class C (Partition walls or backings) – 2.4 MPa
1985-1990	NZS 3102:1983 cited in NZS 4230P:1985 and NZS 4229:1986	12 MPa
1990 - 2004	NZS 4230:1990 and NZS 4210:1989 cited in NZS 4229:1986A1	12 MPa
2004 - 2024	NZS 4210:2001 cited in NZS 4230:2004	12.5 MPa

Table C11.3: Default assumed grout compressive strengths

Period	Default assumed 28 day Grout compressive strength f_g
1948-1985	10.5 MPa or 17.2 MPa if the grout cells in the masonry units are larger than 100 mm in least dimension
1985-2024	17.5 MPa

Table C11.4: Default assumed mortar compressive strengths

Period	Default assumed 28 day compressive strength f_{mortar}
1948-1985	12.4 MPa for all reinforced concrete masonry
1986-1990	12.5 MPa for SED masonry 8.5 MPa for non-SED masonry
1990-2024	12.5 MPa (SED and non-SED)

Mortar compressive strength is not used in filled masonry compressive strength calculations but is included here for completeness.

Note:

Throughout these guidelines, f'_m is used to refer to the probable compressive strength of masonry.

This usage is non-conventional. In most engineering documents, f'_m is used to refer specifically to the specified compressive strength when the masonry reaches a particular age (most often 28 days).

Where assessment of a particular item requires reference to Standards or other documents that define calculation methods based on the specified masonry strength, it is generally acceptable to substitute the probable compressive strength for the purposes of the assessment, unless doing so would contradict provisions of these guidelines.

C11.4.4.3 Probable compressive strength of unfilled masonry block

When part of a partially filled wall, unfilled masonry cells should not be relied upon for compressive strength, instead using only filled cells in compressive strength calculations.

If no cells are filled, then the concrete masonry should not be considered RCM.

Note:

Mortared concrete block without grout is outside the scope of this chapter. The principles of Section C8 may be able to be adopted, but clay brick is typically solid, whereas hollow concrete block may be susceptible to cross web splitting under large axial loads. At a minimum, a significant reduction of wall thickness (to the width of the sum of two face shell thicknesses) is necessary for in-plane strength checks.

C11.4.4.4 Probable elastic modulus

The probable elastic modulus of RCM can be calculated as:

$$E_{\mathrm{m}} = 900 f'_{\mathrm{m}} \quad \text{...C11.2}$$

Note:

RCM is a highly non-linear material, and the effective modulus will vary depending on the magnitude on the loading. It is intended that this equation is used when calculating building period, along with any appropriate reductions of gross section properties as outlined in NZS 3101:2006, Clause C6.9.1.

When considering RCM walls, wall stiffnesses can be calculated via ASCE 41-23 using Equation 11-29.

C11.4.4.5 Probable tensile strength of RCM

The tensile strength of masonry should not generally be relied on when calculating the strength of masonry members. In no situation should the uncracked strength of an element be taken as higher than its cracked strength.

Note:

The tensile strength of masonry is highly variable, and prone to being reduced significantly by shrinkage. Cracking is also in part controlled by the bond strength of the mortar with the concrete block, making its calculation more complex than for reinforced concrete.

C11.4.5 Reinforcing steel

The material properties of reinforcing steel should be obtained from Section C5.

C11.4.6 Reinforcing bar anchorage and development

Where development of reinforcement is critical, specific consideration of the effects discussed in Section C11.4.2 is required. If reasonable confidence can be achieved that voids are not present in the lapping zone, the strength of laps with deformed bars may be calculated following Section C5.4.4, where L_d is taken from Table C11.5.

Where voids are found at critical splices, the required lap length shall be increased by the length of the void along the splice. Splices are a recommended area of focus during site inspection and testing.

Table C11.5: Required reinforcement laps for deformed bars

Steel Grade	Required Lap Length, L_d
275 and 300	40 diameters
380	54 diameters
430	60 diameters
500N and 500E	70 diameters

C11.4.7 Reinforcing bar mechanical couplers and welded splices

Mechanical and welded couplers are uncommon in RCM. If these elements are found in RCM, the principles presented in Section C5 should be adopted.

C11.4.8 Anchorage to RCM elements

Determining the capacity of anchors in RCM elements can be more difficult than their equivalents in either reinforced concrete or URM. This is because the capacity is partially determined by the bond between the face shells of the concrete unit and the grout.

Anchors installed into ungrouted block should not be relied upon for structural loadings without specific site testing.

There are some situations where RCM anchor capacities are documented. When masonry anchors meet all requirements of Appendix C in NZS 4230:2004, the capacities reported in Table C1 in NZS 4230 may be adopted. Similarly, where masonry anchors meet the requirements of NZS 4229:2013 (refer Figures 9.2 to 9.5 in NZS 4229), the capacities reported in Table C1 in NZS 4230 for observation type C masonry may be adopted.

No adjustment to probable capacity should be made for anchor capacities and design strength reduction factors should be used. This approach is consistent with Section C5.4.6 for reinforced concrete.

Testing may be the only available option for critical RCM anchor connections. Testing should be carried out following the requirements outlined in Section C5.4.6 for reinforced concrete anchors.

Note:

The face shell is particularly troublesome when attempting to assess the capacity of masonry anchors. When assessing tension, the strength contribution of the face shell should be taken as 0. When assessing shear, it is considered reasonable to allow bearing stresses up to the compression stress of the concrete block between the anchor and the face shell, providing they are in direct contact (such as when an epoxy grout is used to install the anchor), and capacities are appropriately reduced for any penetrations near the anchor. See Table C11.2 for concrete block compression capacities. This approach is not suitable for ungrouted blocks, as the lack of supporting grout introduces further flexural stresses to the concrete block.

Note:

Assessing engineers would benefit significantly from further research to establish reliable minimum anchor capacities. The research community is encouraged to investigate the capacity of RCM anchors further.

C11.5 Probable Capacities of Beams, Columns and Walls

This section sets out the procedures (with reference to Section C5) for evaluating the probable strength and deformation capacities of beams, columns and walls.

C11.5.1 Strength capacity

The principles and methods provided in Section C5.5 should be followed for RCM with the amendments described below.

C11.5.1.1 General

1. Where the criteria of the standard are met, buildings may be alternatively assessed using the 'bracing schedule' approach from NZS 4229:2013. Further information is provided in Section C11.8.3.

2. Lap splices for RCM should be calculated following Section C11.4.6.

3. For the purposes of assessment, the NZS 4230:2004 definition of a column is adopted. That is: *an element not longer than 790 mm having a minimum width of 240 mm subjected primarily to compressive axial load*. The 'primarily axial load' threshold can be determined following NZS 4230:2004 Clause 7.3.1.5. Longer elements should be assessed as walls.

Note:

For situations where reinforcement continuity and full development can be confidently expected, the performance of RCM should always exceed that of URM. If any element is found to score higher using the appropriate clauses in Section C8 compared with Section C11, then the capacity from Section C8 may be used. When undertaking any RCM assessment using Section C11, the reduced section thickness of hollow concrete block (due to the internal cavities for grout) should be accounted for.

C11.5.1.2 Flexural capacity

1. The direct rotation method should not be used. The empirical data used in the direct rotation method in Section C5 was produced for reinforced concrete, not RCM conditions.

   Note:

   ASCE 41-23 contains methods of assessment that allow more consideration of residual strengths in RCM elements. Section C11.8 covers the use of ASCE 41-23 that meets the intent of these guidelines.

2. The strain limit for unconfined RCM is $\varepsilon_{\mathrm{cm,max}} = 0.003$.

   Note:

   Consideration of the strength and stiffness of foundations should be included in the assessment of RCM elements. Note that due to the scale and proportions of many

RCM buildings, foundation stresses and/or stability will be a limiting mechanism (either overall or in part) due to the foundation stresses induced by seismic lateral loading. Refer to Section C4 and Section C5 for further information and guidance.

3. The strain limit for RCM with confining plates that meet the requirements of Clause 7.4.6.5 in NZS 4230:2004 is $\varepsilon_{\mathrm{cm,max}} = 0.008$.

Note:

Confining plates are an uncommon construction practise and should be confirmed via construction drawings or site inspection.

4. Equally, the strain limit for RCM with steel ties that meet the requirements of Section Cl7.4.6.5 in NZS 4230:2004 is $\varepsilon_{\mathrm{cm,max}} = 0.008$.

5. Flanges should be considered to contribute to the flexural response of a wall if a. is true, and b. or c. are true (see also Figure C11.7Figure C11.7):
   a. Intersecting reinforced bond beams are provided at a vertical spacing not greater than 1.2 m on centre with reinforcement fully developed on each side of the intersection
   b. The face shells of hollow masonry units at the intersection are removed and the intersection is fully grouted;
   c. Units are laid in running bond, and 50% of the masonry units at the intersection are interlocked.

   The length of the contributing flange should be calculated following Section C5.5.2.2.3. Alternatively, flange contributions may be assessed using strut and tie, following the method outlined in Section C5.6.1, substituting the appropriate material properties for f'_c .

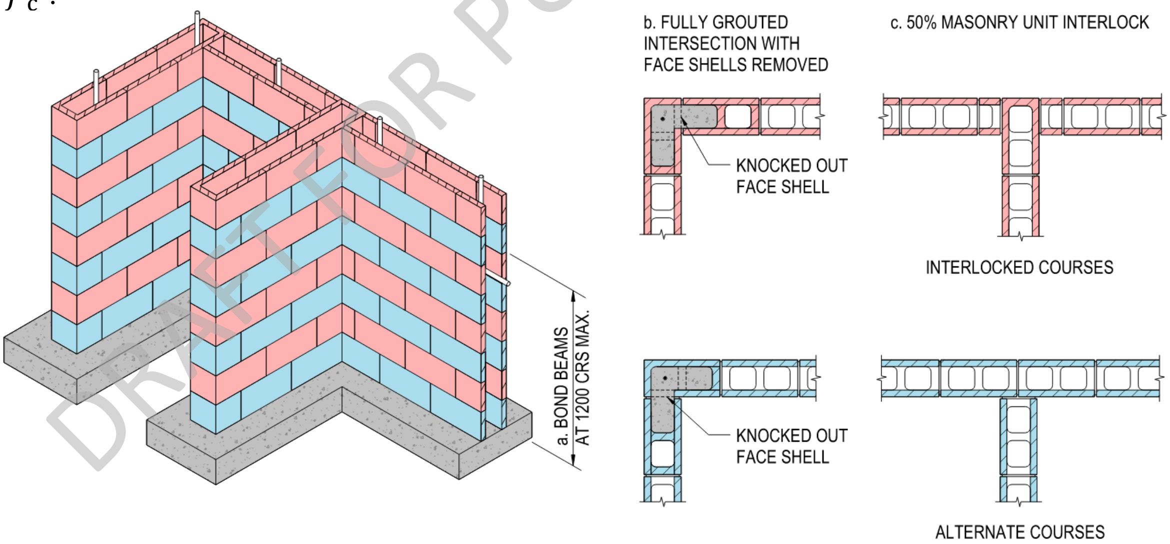


Figure C11.7: Minimum criteria for flange contribution to wall flexural response

6. Unfilled cells in partially filled walls should not contribute to the compressive strength of the wall.

C11.5.1.3 Shear capacity

1. In a partially filled wall, the effective area contributing to the wall shear capacity should be determined from Figure 10.1 of NZS 4230.

2. Sliding shear should be considered as a possible RCM failure mechanism. Sliding shear capacity should be determined using Clause 10.3.2.13 of NZS 4230 using a coefficient of friction, μ_f = 0.7, and the strength reduction factors in Section C11.4.3.

Note:

A recent American review (FEMA P-2208, 2023) included several recommended changes for the assessment of sliding shear in concrete walls (not RCM), which were implemented in ASCE 41-23. The recommendations included lowering the coefficient of friction used when calculating the sliding shear capacity in flexural walls, due to the cracking that occurs at the wall base under cyclic loading (meaning the wall is no longer acting as if it were 'monolithic', even if it was originally cast in one pour). However, walls found to be governed by sliding shear do not suddenly lose gravity support when their capacity is exceeded, and so exhibit more ductility than previously supposed. While RCM is expected to perform similarly to concrete walls, RCM was not explicitly tested. RCM is expected to be less ductile than concrete equivalents and is presently limited to a ductility capacity of 2 for sliding shear, in the absence of specific testing.

3. RCM shear strength should be calculated using NZS 4230:2004 Clause 10.3.2, using the strength reduction factors from Section C11.4.3 and masonry properties as provided in Table C11.6.

Note:

NZS 4230:2004 included a new requirement that horizontal reinforcement must terminate in a standard hook or bend at each end. If a wall does not meet this specific requirement, the shear steel capacity should be calculated using a reduced wall length of d':

$$d' = d - (L_d - L_{dh}) \quad \text{...C11.3}$$

where the above variables are defined in NZS 4230:2004.

Table C11.6: Assessment masonry properties (in place of Table 10.1 in NZS 4230:2004)

Type of stress	RCM Elements where potential plastic hinge zones are checked for grout voids	Other
Compression; f'_{m}	Refer C11.4.2	
Basic shear provided by masonry, general conditions, V_{bm}	$0.2\sqrt{f'_{\mathrm{m}}}$	$0.2\sqrt{f'_{\mathrm{m}}}$
Basic shear provided by masonry in potential plastic hinges of limited ductile structures, V_{bm}[2]	$0.15\sqrt{f'_{\mathrm{m}}}$	0
Basic shear provided by masonry in potential plastic hinges of ductile structures, V_{bm}[2]	0	N/A^1
Maximal total shear, general conditions, V_{g}	$0.45\sqrt{f'_{\mathrm{m}}}$	$0.45\sqrt{f'_{\mathrm{m}}}$

Notes:

1. RCM walls should be considered to have reached their ultimate capacity at limited ductile deformations unless site investigation has checked a sample of potential plastic hinge zones for grout voids.
2. Figure 10.5 of NZS 4230:2004 allows for more granular assessment of f'_{m} as a function of ductility. However, the more granular method should only be used after careful consideration of the detailing in the plastic hinge zone.

C11.5.1.4 Out-of-plane capacity

1. Out-of-plane flexure for fully filled walls may be assessed using standard flexural theory.

2. Out-of-plane flexure for partially filled walls should be assessed using yield line theory. An example of this process can be found in Singh et al. (1999).

3. Out-of-plane flexural capacity may be assessed using the distance between the extreme compression fibre of the unit to the steel centroid. The masonry strength may be taken as f'_{m}.

Note:

RCM walls with typical interstorey (<4 m) heights and reliable connection at each floor are unlikely to be vulnerable to out-of-plane flexure. It is recommended to consider the connection at the top and bottom of each wall segment, as some common construction details can be assessed as having fixed ends.

Note:

The mortar and concrete block compressive strengths are not directly considered in the calculation of flexural out-of-plane strength, despite it being the mortar/concrete block that typically experiences the entirety of the compressive stress. However, the out-of-plane flexural capacity is largely insensitive to selection of f'_{m}. For example, in 15 series block, use of f_{cb} instead of f'_{m} changes the flexural capacity by <10% at typical historical reinforcement ratios. Greater difference may occur for particularly low concrete block strengths (i.e. <10 MPa) and high reinforcement ratios. This approach is also consistent with the examples presented in the New Zealand Masonry Manual (NZCMA, 2012), and by Paulay and Priestley (1992) for out-of-plane wall actions.

However, if the mortar is found to have deteriorated (such as mortar being able to be scraped out with a finger), it is recommended that this is addressed by reducing the thickness of the wall in the out-of-plane calculation.

C11.5.2 Ductility capacity

RCM generally has lesser ductility capacity than reinforced concrete. This is due to multiple factors:

- The strength of RCM is typically less than the strength of reinforced concrete.
- RCM usually does not contain a confined core, which limits the ability of the grout to remain in place after cracking develops.
- Walls with squat aspect ratios also often exhibit a stiff, shear governed response, followed by a quick reduction in force resistance when the shear capacity is exceeded.
- RCM walls often have laps in the plastic hinge zone.

The following factors should be considered when evaluating the ductility of an RCM element:

- Where the thickness of a wall is less than 5% of the clear vertical distance between horizontal lines of support, the adopted wall flexural ductility should not exceed $\mu = 1.25$. This requirement can be relaxed when flanges are present at each end of the wall.
- In beams and columns where bars are found to have inadequate development, and these bars are providing strength to the critical mechanism (i.e. inadequate flexural bar splices when flexurally governed, or inadequate shear steel splices when shear governed), the element should not exceed $\mu = 1.25$.
- Where beam column joints are found to be critical, ductility capacity should not exceed $\mu = 1.25$.
- Flexurally governed walls that are found to have inadequate vertical bar splices at the location of critical flexure should not exceed $\mu = 1.25$, unless it can be shown that the lap splice failure will not cause a significant loss of integrity of the blockwork masonry and grout. If integrity of blockwork masonry and grout can be properly justified, the vertically spliced section should not exceed $\mu = 2$.

Note:

Justification of the blockwork and grout integrity is expected to include an evaluation of what can confine the location of the splice failure. RCM typically does not provide active confinement, which can quickly lead to loss of grout and blockwork masonry under cyclic seismic loading. This rapid loss of materials limits the available ductility capacity.

- Shear governed walls that have inadequate shear reinforcement splices should not exceed $\mu = 1.25$ unless an adequate strut and tie mechanism can be demonstrated. The shear steel may be used as part of the strut and tie, but no load should be allowed to be transferred via inadequate splices.
- Flexural ductility of a wall may be determined using a rational method, such as Section C5 or Clause 2.72 - Clause 2.7.3 of NZCMA (2004). However, walls that are

assessed as fully ductile (i.e. μ > 3) should meet all the detailing requirements of NZS 4230:2004. Flexurally governed walls should not exceed $\mu = 4$.

- Partially grouted walls should not exceed $\mu = 2$. Partially grouted walls should be assessed to identify their critical mechanisms, and the lower ductility limitations of those mechanisms or $\mu = 2$ should be adopted.
- Sliding shear governed walls should not exceed $\mu = 2$.
- Shear governed walls should not exceed $\mu = 1.25$, unless the conditions in the note below are met.

Note:

For RCM walls with:

- squat aspect ratios (length to height ratio ≥ 2),
- low drift demand,
- low axial demand, and
- other lateral structural elements that are still capable of taking shear

a shear failure is unlikely to result in a loss in gravity support, and further lateral system capacity may be available via a SLaMA or pushover analysis. In this situation $\mu = 2$ may be adopted. Where force-displacement curves are produced for $\mu = 2$ shear governed walls, the backbones should model no lateral shear strength beyond the drift when the maximum shear capacity of the wall is first reached. If a wall is instead governed by shear friction, its strength may be maintained until it achieves $\mu = 2$. Each RCM wall should have its drift capacity evaluated individually.

- For out-of-plane flexure a ductility capacity of 2 to 3 (3 being suitable for minimal axial loading) is considered appropriate, providing reinforcement is continuous in zones of potential yielding. For out-of-plane loading, no distinction is made between the ductility capacity of partially grouted and fully grouted walls.
- No specific differentiation is made between round bars and deformed bars in terms of ductility capacity. However it is noted that round bars will likely be found to have insufficient splice lengths, due to the doubling of the required development length in Section C5.4.4.

Note:

Where the phrase 'should not exceed' is used above, it is intended that the ductility capacity will still be calculated by the assessing engineer, then limited to the stated value if it is exceeded.

Note:

Ductility capacities greater than those specified above are possible in certain situations. The limits provided here are considered appropriate to cover many possible configurations. If justification of ductility beyond that stated above is desired by the assessing engineer, the adopted ductility capacity should be supported by experimental data that has similar axial loading, reinforcement, detailing and aspect ratios.

C11.5.3 Retaining walls

RCM is frequently used for retaining walls. Further guidance may be found in Appendix C4B (in Section C4) for retaining wall assessment, including retaining walls that also act as part of a building's lateral load system.

C11.6 Capacities of Diaphragms, Beam-Column Joints and Other Elements

C11.6.1 Strut-and-tie models

Strut-and-tie for RCM should follow Section C5.5.6.1, with substitution of f'_{m} for f'_{c}.

C11.6.2 Beam-column joints

Beam-column joints which meet the confining conditions of NZS 4230:2004 Clause C11.2 may be assessed using Section C5.6.2, with substitution of probable f_{g} (which may be taken as 1.5 times characteristic f_{g}) for f'_{c}. Other beam-column joints may be assessed using NZS 4230:2004 with strength reduction factors from Section C11.4.3, providing they meet the design requirements of the standard.

Note:

NZS 4230:2004 notes that masonry beam-column joints commonly differ from concrete joints due to the lack of confining steel and the common occurrence that all steel is in a single plane (for example, all horizontal bars are placed centrally in the block), so usage of Section C5 is limited here.

Beam-column joints which do not meet the minimum requirements of NZS 4230:2004 should be the subject of special study.

C11.6.3 Diaphragms

RCM structures typically use either reinforced concrete or timber diaphragms. Reinforced concrete diaphragms should be assessed following Section C5.6.3. For timber diaphragms, assessment should include Section C8.8.3 (which covers timber diaphragms for URM systems but is also relevant to RCM). Note that diaphragm stiffness may significantly affect load distribution to the various RCM lateral load resisting elements.

Note:

Bond beams can horizontally transfer out-of-plane wall load where diaphragms are not present. Table 10.1 in NZS 4229:2013 can be a useful source for maximum bond beam spans for walls which meet the detailing and scope requirements of the standard.

C11.6.4 Support of precast units

Refer to Section C5E.4.1 when RCM face shells support precast elements.

C11.7 Global Capacity of RCM Buildings

The displacement capacity of the global system should be checked following the guidance of Section C5.7.

C11.8 Alternative RCM Capacity Techniques

C11.8.1 Use of alternative codes

Two alternative means of assessing RCM are considered to meet the intents of these guidelines provided their respective criteria are met, as outlined in Section C11.8.2 and Section C11.8.4.

It is intended that if an alternative standard is used to calculate a capacity, the methods of the standard shall be used in their entirety for the assessed element. In other words, 'picking and choosing' of parameters between various standards does not meet the intent of these guidelines. It is further intended that alternative standards will use latest version of said standard.

C11.8.2 RCM assessment using NZS 4230:2004

RCM may be alternatively assessed following NZS 4230:2004, provided that the detailing requirements of the standard are met. This approach may use the strength reduction factors from Section C11.4.3, and the material properties of Sections C11.4.4 and C11.4.5. When calculating wall shear steel contributions, if the horizontal steel does not have hooked ends, the effective depth should be reduced as described in Equation C11.3.

C11.8.3 RCM assessment using NZS 4229:2013

Buildings that are consistent with the scope of NZS 4229:2013 (refer Clause 1.1.3 of NZS 4229:2013), may be assessed via a 'bracing schedule' approach as per Clause 5 of NZS 4229:2013. The standard includes requirements in terms of scale, reinforcement content, axial loadings, detailing, and redundancy of wall layouts. This standard assumes that a displacement ductility of $\mu = 2.0$ is available, without requiring specific identification of plastic hinge zones or explicit capacity design. Refer to NZS 4229:2013 including commentary for further information.

When using NZS 4229:2013 to assess existing structures, the stated capacities in the standard should be used without scaling the capacities from probable to characteristic strengths, and without removal of strength reduction factors.

C11.8.4 RCM assessment using ASCE 41-23

RCM may be alternatively assessed following ASCE 41-23, using the lower material strength from Section C11.4 and ASCE 41-23. It is noted that ASCE 41-23 allows greater consideration of the post-yield capacity of walls, so may result in improved scores for RCM. Even when using ASCE 41-23, it is intended that demand calculations will still follow Section C2. When using ASCE 41-23, displacement capacity should be limited to γ_m for both flexurally governed and shear governed walls.

C11.9 Improving the Seismic Performance of Reinforced Concrete Masonry

RCM strengthening techniques and approaches are similar to that provided for reinforced concrete (usually reinforced concrete walls) in Section C5.8. Similarities are also found with URM strengthening techniques (Section C8.12).

In some situations, the need for extensive strengthening can be avoided by creating higher ductility mechanisms such as wall rocking and/or foundation yielding. This can be achieved via selected strengthening and even targeted weakening to create a desirable strength hierarchy.

Strengthening priorities are dependent on the specifics of the building, but a general strengthening priority checklist includes:

- securing/strengthening any appendages such as cantilever parapets and any elements with excessive axial loadings
- improving overall structural integrity including face loaded connections, wall-to-diaphragm connections and foundation integrity
- strengthening transfer diaphragms
- strengthening wall elements in-plane, prioritising those elements subjected to high shear and axial demands. Creating reliable flexural yielding mechanisms is generally desirable where wall aspect ratios permit
- strengthening walls out-of-plane.

The common squat geometry of RCM walls means that the ability to improve displacement capacity is frequently limited. Where a new structure is added, its load carrying capacity should be considered relative to the deformation capacity of the existing structure. For example, a flexible steel frame will not take significant load prior to the loss of lateral structural capacity in an adjacent squat RCM wall. Assessing the combined structural system can be assessed using SLaMA, or other displacement-based assessment methods. The load paths through existing diaphragms and foundations should also be explicitly considered when adding new structural elements to an existing system.

Common strengthening options for blockwork include:

- grout filling (typically via pumping) unfilled hollow blocks to improve shear capacity
- chasing existing blockwork to install vertical and horizontal reinforcement. Chasing is frequently filled in using high-strength, shrinkage-compensated grouts and/or pressure injection of epoxy. Note that the stability of these walls in the temporary case can be an issue, and there are practical limitations for both grout workability and achievable epoxy volumes
- adding sprayed reinforced concrete cast against and tied into the face of existing blockwork (shotcrete)
- vertically post tensioning walls
- steel, reinforced concrete or FRP confinement of heavily axially loaded elements
- cantilever steel columns (also known as ‘strong backs’) are a common addition when cantilever walls have inadequate out-of-plane capacity

- RCM fire walls are frequently only accessible on one side. Out-of-plane strengthening commonly involves steel strong backs or perpendicular steel moment frames. Horizontal steel collectors may also be necessary.

Other considerations include:

- It is prudent to consider the magnitude of forces on connections early in strengthening design. More numerous, smaller demand connections are frequently found to be more efficient than smaller numbers of connections with high loads. This is due to the low bolt strengths achieved when bolting into RCM, and the further capacity reductions that arise from many bolts in a small area (bolt group effects). These considerations may end up driving the type and distribution of strengthening elements required.
- Conceptual design of connections can be based on the fixing capacities provided in Table C1 in Appendix C of NZS 4230:2004 (approximately 10-20 kN per bolt at approximately 200-300 mm centres). Subsequent design can involve matching the detailing requirements for the NZS 4230 anchors, or be confirmed via proprietary anchor software.
- NZS 4229:2013 provides details that can be useful starting points for strengthening connections to diaphragms. For example, refer Clause 9.2.6.2 and Clause 9.3.4.2 in NZS 4229. Note post installed anchors will achieve reduced bolt capacities in shear and tension.
- Bolting through block walls is considered preferable to chemical anchoring where possible, as bolting through provides greater tension capacity.
- Note that the 'Simplified Capacity Design Approach' as outlined in NZS 4230:2004 may offer an expedient approach to strengthening (and assessment), for buildings that meet the limitations of that clause.
- The *NZ Concrete Masonry Manual* (NZCMA, 2012) and the *User's Guide to NZS 4230:2004* (NZCMA, 2004) both contain useful information that may aid designers. Web links (current at the time of publication) to these resources are included in the references at the end of this section.

References

ASCE 41-23 (2023). *Seismic Evaluation and Retrofit of Existing Buildings*, American Society of Civil Engineers, Reston, Virginia, USA.

Cathie, M. (2003). *NZS 4230 Review design of reinforced concrete masonry structures,* New Zealand Concrete Society Concrete Technical Conference, Taupo, NZ.

Davenport, P. (2004). *Review of seismic provisions of historic New Zealand loading codes*, New Zealand Society of Earthquake Engineering (NZSEE) Conference, Rotorua, NZ.

FEMA 306 (1998). *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings*, Applied Technology Council, Redwood City, California, USA.

FEMA P-2208 (2023). *NEHRP Recommended Revisions to ASCE/SEI 41-17, Seismic Evaluation and Retrofit of Existing Buildings* (FEMA P-2208). Federal Emergency Management Agency, Washington, D.C., 710p.

Fikri, R., Dizhur, D. & Ingham, J. (2019). *Typological study and statistical assessment of parameters influencing earthquake vulnerability of commercial RCFMI buildings in New Zealand.* Bulletin of Earthquake Engineering, Vol. 17, No. 4, 2011–2036. https://doi.org/10.1007/s10518-018-00523-x.

Gaerty, L. (1991) *Concrete Masonry Design Code NZS 4230:1990*, New Zealand Concrete Society Concrete Technical Conference, Taupo, NZ.

Isaacs, N. (2015). *Hollow concrete blocks in New Zealand 1904-10*, Construction History, Vol. 30, No. 1, 93-108.

Isaacs, N. (2022). *NZSS 1900 Model Building By‐law: a planned evolution*, AHA: Architectural History Aotearoa Vol. 19: 100-110.

NZCMA (2004). *User's Guide to NZS 4230:2004*, New Zealand Concrete Masonry Association Inc. July 2004. https://cdn.ymaws.com/concretenz.org.nz/resource/resmgr/docs/nzcma/nzcma_ms02.pdf

NZCMA (2012). *New Zealand Concrete Masonry Manual*, New Zealand Concrete Masonry Association Inc. July 2012.
https://cdn.ymaws.com/concretenz.org.nz/resource/resmgr/docs/masonry/mm_4.1_design_of_reinforced.pdf

NZS 3101 (2006). *Concrete Structures Standard*, NZS 3101:2006, Standards New Zealand, Wellington, NZ.

NZS 4210 (2001). *Masonry Construction: Materials and Workmanship* NZS 4210:2001, Standards New Zealand, Wellington, NZ.

NZS 4229 (1999). *Concrete Masonry Buildings Not Requiring Specific Engineering Design*, NZS 4229:1999, Standards New Zealand, Wellington, NZ.

NZS 4229 (2013). *Concrete Masonry Buildings Not Requiring Specific Engineering Design*, NZS 4229:2013, Standards New Zealand, Wellington, NZ.

NZS 4230 (2004). *Design of Reinforced Concrete Masonry Structures*, NZS 4230:2004, Standards New Zealand, Wellington, NZ.

Paulay, T. and Priestley, M.J.N. (1992). *Seismic design of reinforced concrete and masonry buildings*, John Wiley and Sons, New York.

Priestley, M.J.N. (1985). *Seismic design of masonry structures to the new provisional New Zealand Standard NZS 4230P*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 18, No. 1, 421-443, March 1985.

Singh, S.S., Cooke, N. and Bull, D.K. (1999). *Out-of-Plane Performance of a Partially Filled Reinforced Concrete Masonry Wall with Ribraft Floor,* Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 32, No. 2, 90-101, June 1999.

Smith, P. and Devine, J. (2011). *Review of Masonry Standards in New Zealand,* Canterbury Earthquakes Royal Commission, Record ID ENG.HOL.0002A.

Wylie, T. (1993). *Summary of Masonry Design Codes*, New Zealand Concrete Society Concrete Technical Conference, Auckland, NZ.

Zhang, X. (1998). *Out-of-plane Performance of Partially Grouted Reinforced Concrete Masonry Walls under Simulated Seismic Loading*, Master's Thesis, University of Canterbury.

Appendix C11A Test Methods for Investigating Material Properties

C11A.1 Reinforced Concrete Masonry

The following table adapts the testing table found in Section C5's appendices for suitability to RCM when investigating concrete masonry material properties.

New tools are emerging in the market that can compensate for some of the listed drawbacks by using multiple testing techniques.

Table C11A.1: Overview of destructive, semi-destructive and non-destructive tests for investigating concrete masonry material properties

Method	Capability/Use	Advantages	Disadvantages
DESTRUCTIVE TESTS			
Compressive test	Strength of concrete masonry	Direct evaluation of concrete masonry strength from compressive tests on specimens	Testing of concrete masonry requires a large sample including multiple concrete blocks to capture the effects of the various components. The sample also needs to be free of reinforcing bar Making good works needed to replace the test sample
SEMI-DESTRUCTIVE TESTS			
Mortar scratch test	Assessment of masonry strength	Minimal tools needed, can be checked with a fingernail and aluminum pick	Qualitative only Mortar strength does not affect the combined RCM strength unless it has significantly degraded
Borescope/ fibrescope	To check the extent of cavities	Direct visual inspection of inaccessible parts of an element Cheap borescopes are available in the NZ marketplace	Needs additional fibre to carry light from an external source (although some tools now have built-in light sources) Requires drill holes or other openings to use
NON-DESTRUCTIVE TESTS			
Visual tests	The first step in investigating RCM	Quick evaluation of visible damage, such as cracking, spalling, mortar and concrete block deterioration	No detailed information
Electromagnetic/ ferro scanning	Check for presence of reinforcing steel	Relatively inexpensive, non-destructive Depth to steel can be reported	Devices generally need regular calibration and can provide inconsistent results While thickness of reinforcing bars is claimed in the marketing, differentiating between D12 and D16 bars is often difficult False positives can occur

Method	Capability/Use	Advantages	Disadvantages
Ultrasonic pulse velocity	Evaluation of concrete strength and quality Identification of internal damage/voids and location of reinforcement	Excellent for determining the quality and uniformity of concrete masonry; especially for rapid survey of large areas and thick members	Access is needed to both faces Requires experienced operator The test requires smooth surfaces for a good adhesion of the probes
Ultrasonic echo method	Detecting presence or absence of voids	Access to only one face is needed Internal discontinuities and their sizes can be estimated	Requires experienced operator
Impact echo method	Detecting presence or absence of voids	Access to only one face is needed	Requires experienced operator
Gamma radiography	Location of internal cracks, voids and variations in density of cementitious material	Simple to operate Applicable to a variety of materials	X-ray equipment is bulky and expensive Difficult to identify cracks perpendicular to radiation beam Uncommon tool for RCM assessment
CT scanning	Imaging/mapping	3D crack/damage monitoring	Sophisticated software for analysis Not in situ application Access to CT scanner needed Uncommon tool for RCM assessment
Infrared thermography	Detecting presence of filled and unfilled cells in concrete blocks. Refer Figure C11A-1	Permanent records can be made Tests can be done without direct access to surface by means of infrared cameras	Sensitive to thermal interference from other heat sources The depth and thickness of subsurface anomalies cannot be measured
Ground penetrating radar (GPR)	Identification of location of reinforcement, depth of cover, location of voids and cracks Determination of in situ density and moisture content	Can survey large areas rapidly Suitable for some common block thicknesses (15 series, 20 series)	Low level signals from targets as depth increases Experience needed to interpret results Opening up works occasionally needed to confirm results
Acoustic emission	Real time monitoring of concrete degradation growth and structural performance	A few transducers are enough to locate defects over large areas Can detect the initiation and growth of cracks in concrete masonry under stress	Passive technique, could be used when the structure is under loading Uncommon tool for RCM assessment
Ultrasonic tomography (MIRA)	Uses high frequency (greater than 20,000 Hz) sound waves to characterise the properties of materials or detect their defects	Thickness measurement, reinforcement location, and distress evaluation	Significant efforts and user expertise are required for measurement and data interpretation of large scale application

Method	Capability/Use	Advantages	Disadvantages
Petrography	Forensic investigation of concrete masonry Determining the composition and identifying the source of the materials Determining the depth of fire damage	Microscopic examination of samples	Laboratory facilities as well as highly experienced personnel are needed to interpret the result Uncommon tool for RCM assessment

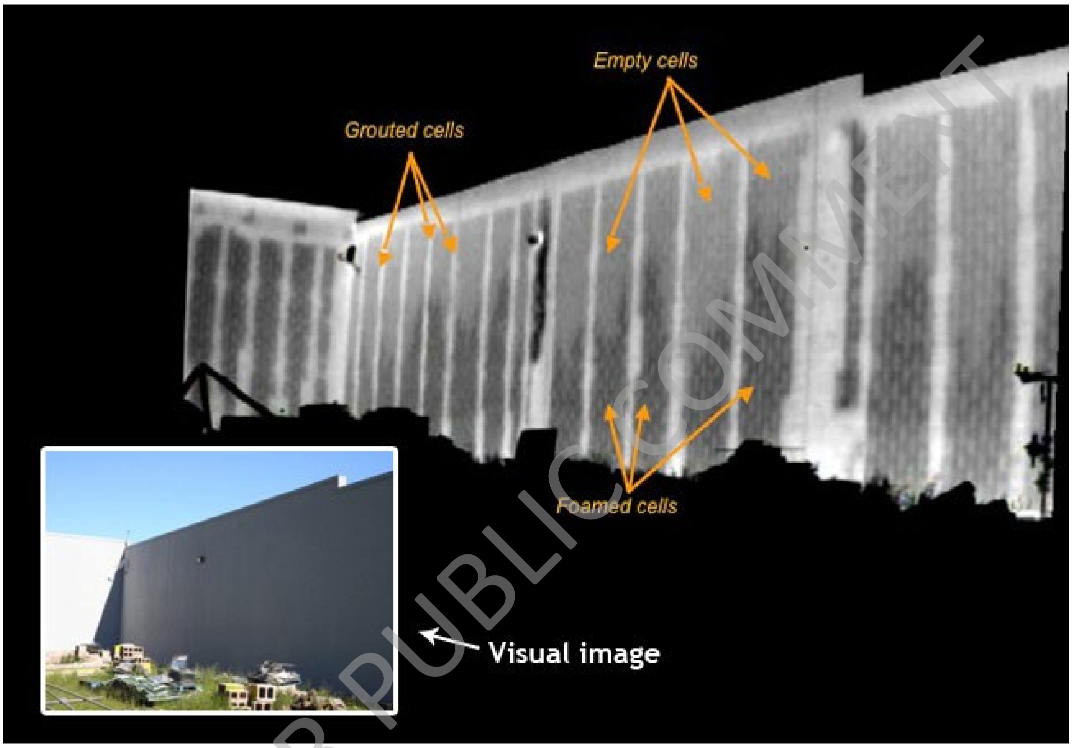


Figure C11A.1: Example of infrared thermography showing filled and unfilled block. Source image: www.infraredimagingservices.com

C11A.2 Reinforcing Steel

Steel testing methods are unchanged from the appendices of Section C5.