Seismic Considerations for the Art Deco Interwar Reinforced Concrete Buildings of Napier, New Zealand

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February 2015

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Walsh, K., Elwood, K., and Ingham, J. (2014). "Seismic Considerations for the Art Deco Interwar Reinforced-Concrete Buildings of Napier, New Zealand." *Nat. Hazards Rev.*, 10.1061/(ASCE)NH.1527-6996.0000169, 04014035.

ABSTRACT

Following the devastating 1931 Hawke's Bay earthquake, buildings in Napier and surrounding areas in the Hawke's Bay region were rebuilt in a comparatively homogenous structural and architectural style comprising the region's famous Art Deco stock. These interwar buildings are most often composed of reinforced concrete two-way space frames, and although they have comparatively ductile detailing for their date of construction, are often expected to be brittle, earthquake-prone buildings in preliminary seismic assessments. Furthermore, the likelihood of global collapse of an RC building during a design-level earthquake became an issue warranting particular attention following the collapse of multiple RC buildings in the February 22, 2011 Christchurch earthquake. Those who value the architectural heritage and future use of these iconic Art Deco buildings—including building owners, tenants, and city officials, among others—must consider how they can be best preserved and utilized functionally given the especially pressing implications of relevant safety, regulatory, and economic factors. This study was intended to provide information on the seismic hazard, geometric weaknesses, collapse hazards, material properties, structural detailing, empirically based vulnerability, and recommended analysis approaches particular to Art Deco buildings in Hawke's Bay as a resource for professional structural engineers tasked with seismic assessments and retrofit designs for these buildings. The observed satisfactory performance of similar low-rise, ostensibly brittle RC buildings in other earthquakes and the examination of the structural redundancy and expected column drift capacities in these buildings, led to the conclusion that the seismic capacity of these buildings is generally underrated in simple, force-based assessments.

KEYWORDS

Reinforced concrete; Art Deco; Napier; Earthquake; Hawke's Bay; Initial evaluation procedure, Nonductile.

INTRODUCTION

Early RC buildings are an important part of the architectural heritage of relatively young countries like New Zealand that historically had limited access to structural iron and steel. Despite having been constructed within the past century, the famous RC Art Deco buildings in Hawke's Bay, New Zealand, represent a critical stock of architectural heritage for a country that has a proud history of structural engineering with RC and of exhibiting resilience to natural disasters. All nonresidential buildings constructed in Napier during the interwar period (i.e., 1920–1940) and remaining today are considered part of Napier's Art Deco building stock, regardless of actual architectural style; although most of these buildings have some elements of the Art Deco aesthetic. Art Deco was an architectural style popularized during the 1920s and 1930s, defined by opulent colors and discrete, rectilinear geometric shapes (McGregor 1998). In the city of Napier alone, 140 Art Deco nonresidential buildings existed at the end of 2012 (McGregor 2012) with at least 58 of these buildings registered with the New Zealand Historic Places Trust (NZHPT) (2012) and at least 123 Art Deco buildings included on local registries for historic preservation (City of Napier 2001, 2011). Furthermore, in the nearby city of Hastings, more than 150 nonresidential buildings—at least 90 constructed of RC—have been identified as having been constructed in a similar time period (G. Lethbridge, unpublished internal report, September 30, 2013).

Tourism stemming largely from Napier's Art Deco attractions contributes greatly to the Hawke's Bay region's revenue (Stewart 2009), and Napier's Art Deco buildings are of immense value to the cultural and civic heritage of the Hawke's Bay community. However, a lack of understanding of the expected performance of these buildings in an earthquake threatens their continued utility. Past engineering assessments of these ostensibly brittle RC low-rise buildings have predicted generally poor seismic performance (e.g., van de Vorstenbosch et al. 2002), contrary to the empirical evidence from the 1931 Hawke's Bay earthquake (Mitchell 1931; Brodie and Harris 1933) and empirical evidence from other historical earthquakes in New Zealand (Dowrick and Rhoades 2000). These buildings are now threatened with forced vacancy or demolition by legislation (New Zealand Parliament 2004) dependent on their estimated seismic capacities. As a result, other researchers have called for more sophisticated studies into the seismic capacities of Art Deco buildings in Hawke's Bay (Dowrick 2006).

RESEARCH MOTIVATION

As noted, simplified seismic assessment methods, such as qualitative, preliminary procedures recommended by the New Zealand Society for Earthquake Engineering (NZSEE 2006) or even equivalent static analyses, may lead to distinct underestimation of the likely seismic performance of many of Hawke's Bay's Art Deco buildings. Because of the high costs and lead times associated with commissioning quantitative finite-element model-based detailed seismic assessments for every Art Deco building in the region, a study undertaken to demonstrate that widespread typological and geometric surveys of the region's Art Deco building stock would provide the following advantages:

1. Immediate guidance to local officials responsible for mitigating the impacts of regional hazards and to Art Deco building owners and occupants on the general seismic risk associated with these buildings: Understanding the typological characteristics of these buildings and the type and extent of damage observed in similar buildings damaged by earthquakes elsewhere can help communities better understand their resilience to major earthquakes and help them prepare for losses in service, even if life-safety is not compromised;
2. Guidance to engineers on what vulnerabilities, geometries, material strengths, and reinforcement detailing have been identified within the building stock so as to improve the accuracy and consistency of detailed models and reporting from practitioners;
3. Guidance for subsequent finite-element model-based detailed seismic assessments of prototypical Art Deco buildings such that the results can be provisionally extrapolated to a large proportion of Art Deco buildings in the region and provide consulting engineers assessing individual buildings with a reference point for comparison; and
4. Demonstrate the utility of typological investigations for assessments of presumably vulnerable, heritage building stocks.

REGIONAL SEISMICITY AND SEISMIC DESIGN REQUIREMENTS

The relative location and seismic hazard of Napier-Hastings compared with other major urban centers in New Zealand are illustrated in Fig. 1. The color spectrum at the bottom of Fig. 1 indicates an increase in seismic hazard from left to right as measured by the expected peak ground acceleration (PGA) at a site with shallow subsoil caused by an earthquake with an average return period of 475 years. The seismic hazard in Napier-Hastings is approximately three times the seismic hazard for Auckland (New Zealand's largest city) and nearly equal to the seismic hazard in Wellington (New Zealand's capital city with the highest seismic hazard in the country among major urban centers).

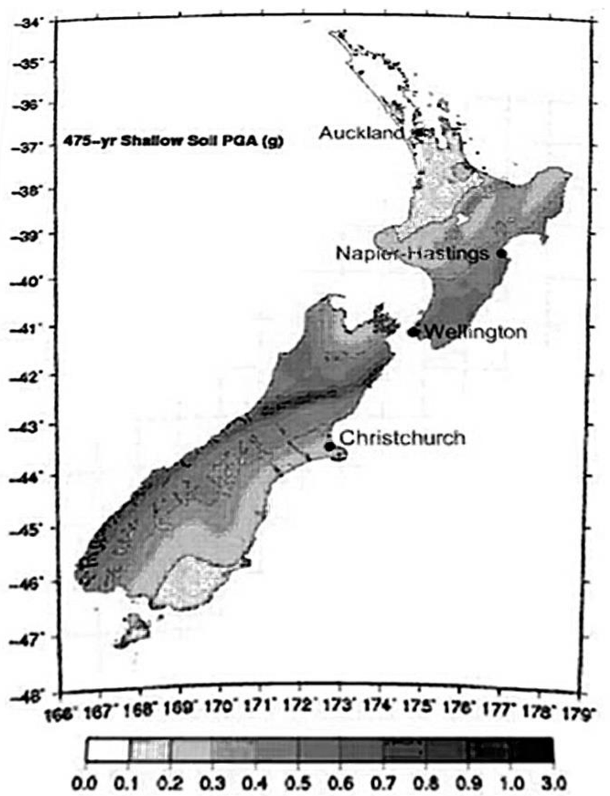


Fig. 1. Referenced New Zealand cities superimposed on seismic hazard map (adopted from Stirling et al. 2012)

Design requirements in New Zealand [Standards New Zealand (NZS) 2002] prescribe that buildings subjected to design basis earthquake (DBE) actions be designed for "avoidance of collapse of the structural system… or parts of the structure… representing a hazard to human life inside and outside the structure… [and] avoidance of damage to non-structural systems necessary for… evacuation." In accordance with the seismic assessment guidelines published by NZSEE (2006), the emphasized performance level considered in the assessments discussed in this paper is the ultimate limit state (ULS), which is theoretically equivalent to the life safety (LS) performance level considered in ASCE 41-13 (ASCE 2014).

TYPOLOGICAL STUDY OF NAPIER'S EXISTING ART DECO BUILDINGS

Due to the confined time period in which so many of the Art Deco buildings in Napier's city center were constructed, the structural and architectural styling is distinctively consistent. Although not all of the Napier Art Deco buildings are formally or completely Art Deco in style, as Spanish Mission, Stripped Classical, Chicago School, Prairie School, Art Noveau, Beaux Arts, and other architectural styles are distributed among the building population in Napier (McGregor 2012), the term Art Deco buildings is commonly used to refer to all of the nonresidential interwar (i.e., 1920–1940) buildings in the region. Examples of the four most prominent Napier Art Deco architectural styles are shown in Fig. 2.

Of the existing Art Deco buildings in Napier, 125 were identified by amalgamating information from a number of sources (McGregor 1998, 2003, 2012; City of Napier 2001, 2011; Shaw and Hallett 2002; Bilman et al. 2004; Stewart 2009; New Zealand Historic Places Trust 2012). Fig. 3 shows a graphical representation of construction years for this building stock; notably, 84% of the buildings were constructed or reconstructed soon after the 1931 Hawke's Bay earthquake (1931–1936). To identify patterns to guide more detailed levels of analyses, some common traits from these 125 existing Art Deco buildings were cataloged, including principal construction material, number of stories, footprint geometry relative to neighboring buildings, principal architectural style, and principal architect. The prototypical Art Deco building (i.e., one associated with the dominant traits shown in Figs. 3 and 4) was constructed soon after the 1931 earthquake, is comprised of an RC frame, is two stories in height, is a row style building relative to its neighbors, is formally Art Deco in architectural style, and was designed by one of five firms.

(a) (b) (c) (d)

Fig. 2. Examples of architectural styles in Hawke's Bay's Art Deco building stock: (a) Daily Telegraph building is of formal Art Deco style; (b) Criterion Hotel is of Spanish mission style; (c) Bennett's building is of stripped classical style; (d) wine center (former AMP Building) is of Chicago school/prairie style (images by Kevin Q. Walsh)

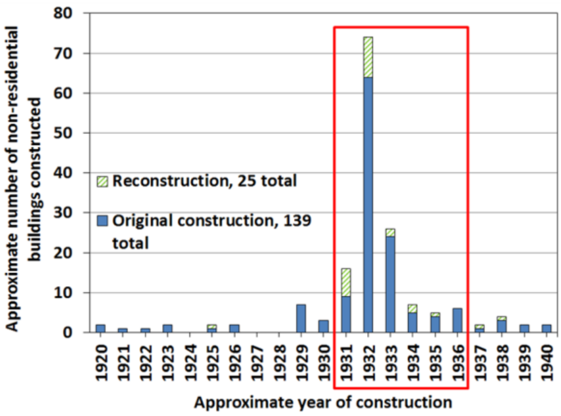


Fig. 3. Years of construction and reconstruction for Napier's Art Deco building stock

The literary sources used to collect data on the Art deco building stock of Napier were rarely specific about whether walls or frames were constructed. However, based on knowledge of architectural preferences in New Zealand at the time and the researchers' observations from a more detailed investigation of seven representative buildings (to be discussed further in a subsequent section), it appears that at least 90% of the buildings identified in this stock were mostly constructed of RC and were built primarily as two-way space moment-resisting frames (i.e., regular column spacing on both the perimeter and interior, often with beams running in both orthogonal directions). RC shear walls are also present in some buildings (van de Vorstenbosch et al. 2002). RC slabs (cast contiguously with the RC frames) appear to dominate the stock's structural diaphragms, although there appears to be some notable variation in roof diaphragm construction (timber, concrete, and corrugated galvanized iron) as well as the inclusion of steel framing (partial framing, retrofitting, or composite with concrete) supplementary to the RC framing in some buildings.

Trends related to the basic footprint geometry of Napier's Art Deco buildings are illustrated in Fig. 4(a). A row internal building has no measurable separation gap between it and its neighbors on either of its two side walls [e.g., Fig. 2(c)]. A row-end building is abutted by a neighboring building on only one wall. A row-corner building is abutted by neighboring buildings on two walls that are orthogonal to each other. Finally, an isolated building has some measurable gap between neighboring buildings on all sides. RC row internal buildings either one or two stories in height constitute almost half of the 125 building surveyed in this typological study [Fig. 4(b)]. Art Deco buildings in Napier with three stories above grade are only estimated to constitute approximately 5% of the building stock.

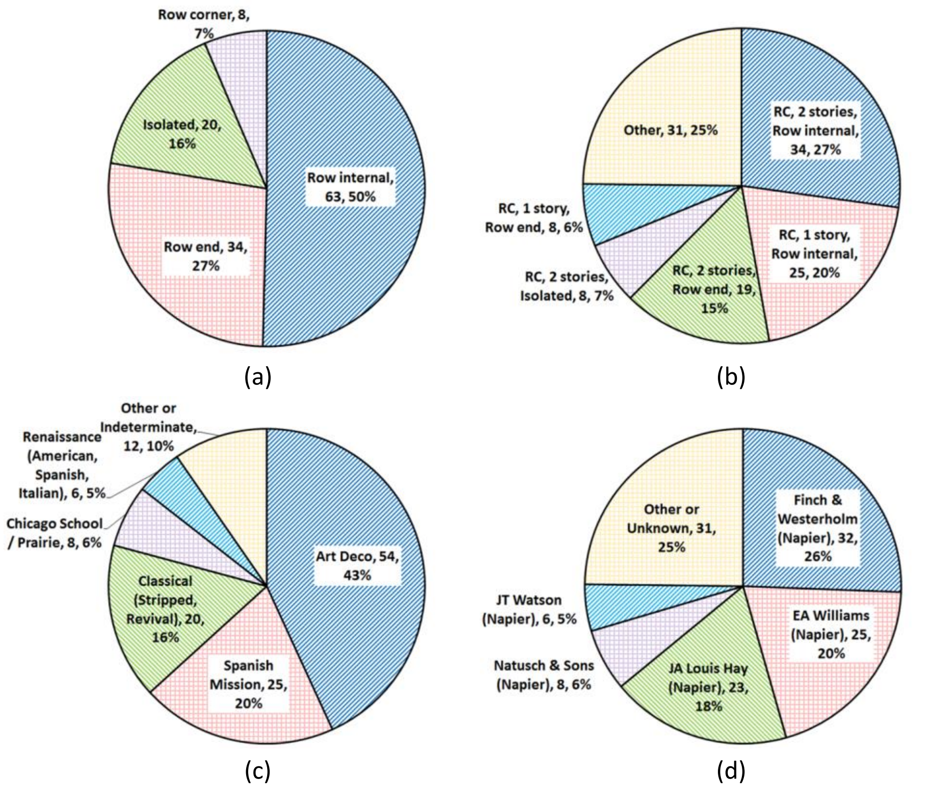


Fig. 4. Traits of Napier's 1920–1940 nonresidential building stock remaining in 2012 with representative number and percentage of buildings, respectively, associated with each trait: (a) footprint relative to neighboring buildings; (b) prominent typological groupings; (c) principal architectural styles; (d) principal architects

Architects in Hawke's Bay during the interwar period usually produced both the architectural and structural drawings. These architects often replicated numerous plan details for reconstructed Napier Art Deco buildings to produce the plans quickly enough for post-disaster building reconstruction, often resulting in similar or identical column geometry and detailing being implemented in buildings designed by the same architect. Hence, the identification of the style and architect [Figs. 4(c and d)] is important for the typological structural assessments exercise; buildings of similar architectural styles or designed by the same architect are considered much more likely to share structural similarities, and they may also share common falling hazards such as high parapets or exterior ornamentation.

GEOMETRIC STUDY OF NAPIER'S EXISTING ART DECO BUILDINGS

A field investigation was undertaken to document geometric measurements, irregularities, and other observations of building exteriors for 109 Art Deco buildings in Napier (constituting 78% of the estimated 140 Napier Art Deco buildings still existing as of 2012). Most buildings had only one visible or accessible wall (in accordance with the prominence of row buildings in the stock). Nonetheless, surveys were able to be made of multiple side walls on some buildings, resulting in 139 total building sides being surveyed from the 109 buildings.

Geometric irregularities

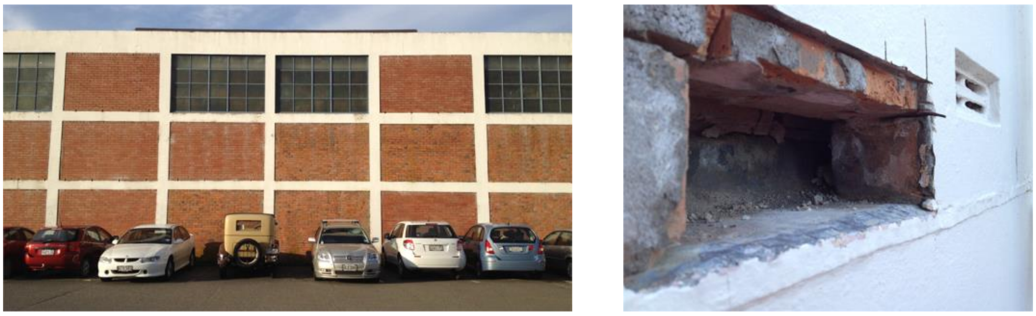
The 109 buildings investigated in the geometric study were assessed for potential critical structural weaknesses (NZSEE 2006) resulting primarily from geometric irregularities. The Napier Art Deco building stock is generally comprised of rectangular and regularly configured buildings, such that severe geometric and configuration irregularities are not commonplace. In accordance with the criteria associated with the NZSEE (2006) initial evaluation procedure (IEP), fewer than 5% of all buildings surveyed were determined to have potential plan or vertical irregularities (expected to cause significantly detrimental eccentric deformations when subjected to lateral load) or potentially shortened columns (expected to result in brittle column behavior). Due to the density of the built environment in the Napier City Center, the most significant potential structural weakness identified in the building stock was the potential for pounding (the phenomenon of neighboring buildings impacting each other off-phase and/or at offset story heights when subjected to seismic forces). Unlike with more historical unreinforced masonry (URM) row construction in New Zealand, where neighboring buildings often shared load-bearing walls, it appears that most Art Deco buildings were constructed with separate load-bearing exterior frames (i.e., isolated column lines) between neighboring buildings, such that the buildings are expected to oscillate independently during an earthquake aside from pounding restraints. NZSEE (2006) instructs engineers carrying out the IEP to reduce the estimated capacity for a building by at least 30% if the separation gap between two buildings is less than 1/200 of the building height. The percentage of Napier Art Deco buildings judged to have at least one wall with less separation from the neighboring building than 1/200 of the building height is 84% [all cases in Fig. 4(a) besides isolated]. However, the prescribed strength reduction in the IEP associated with pounding potential is likely to be conservative for most buildings owing to the qualitative and provisional nature of the procedure. From the geometric survey, it was determined that only 12% of Napier Art Deco buildings had elevated potential for pounding because of excessive height differences and/or offset story alignment between neighboring buildings. Severe pounding potential is most common in situations in which an Art Deco building neighbors a more modern building, and it is a critical aspect when considering the expected damage level and collapse potential of select buildings in earthquakes, as observed in the 2011 Christchurch earthquake (Cole et al. 2012).

Non-structural life-safety hazards

In many cases the most significant hazards to people during an earthquake may not be the failures of load-bearing structural elements but rather the collapse of nonstructural parts and components. Although Napier's Art Deco building stock has few tall chimneys and gable end walls, slender RC parapets and unreinforced masonry infill walls are prominent. These components can be especially dangerous to pedestrians just outside a building (Ingham and Griffith 2011; Cooper et al. 2012).

Approximately 30% of buildings surveyed in the geometric study had parapets. The average parapet height above the roof diaphragm was 1.4 m, and the average parapet length between building corners or other return walls was 11.9 m (the average parapet length is indicative of the average Art Deco building plan dimension parallel to the street based on typical construction practices). Information pertaining to parapet thickness and steel reinforcement detailing was not able to be procured during the geometric survey. However, intrusive investigations were performed separately on the parapets of two Art Deco buildings, with one each located in Hastings and Napier. The Art Deco building in Hastings had parapet heights ranging from 0.79 m along the backside of the building to 2.03 m at the prominent corners of the building on the street front. The parapet was consistently 0.30 m thick with one reinforcement layer of 12.7-mm-diameter round steel bars spaced at approximately 0.60 m both horizontally and vertically. Atop a three-story building, the tallest sections of the parapet were assessed as having a capacity less than 20% of the ULS demands. The Art Deco building in Napier had parapet heights ranging from 0.92 m along the backside of the building to 1.80 m along the street front. The parapet was consistently 0.15 m thick with one reinforcement layer of 12.7-mm-diameter round steel bars spaced at approximately 0.30 m both horizontally and vertically. Also atop a three-story building, the tallest sections of the parapet were assessed as having a capacity less than 50% of the ULS demands. Investigations utilizing intrusive or scanning methods are often needed to assess parapets in these building because it is often unclear from the available plans whether appropriate anchorage to the frames below were provided for the tall parapet walls on the buildings inspected in Hawke's Bay.

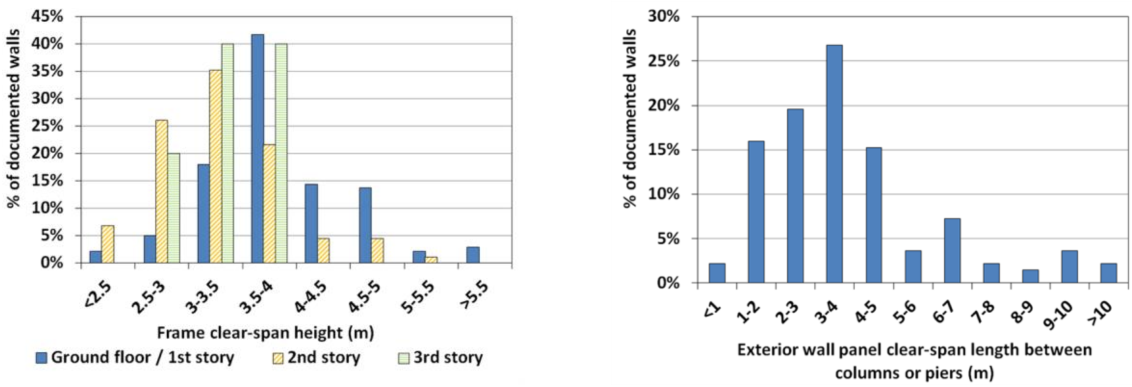
Of the 150 contemporary nonresidential buildings identified in Hastings, at least 40% were identified as having clay brick URM infill panels (G. Lethbridge, unpublished internal report, 2013). In Napier, many of the perimeter frames surveyed as part of the Napier Art Deco geometric study were infilled with either RC or clay-fired brick URM in addition to containing portal openings for doors and windows [Fig. 5(a)]. Although not always apparent because of architectural plaster and ornamentation, it appeared that approximately 50% of the 109 buildings surveyed had some perimeter frames at least partially infilled with URM. Of those buildings identified as having URM infill, approximately 70% were identified as having URM infill panels with cavities, or air gaps, separating two single wythes of brick [each wythe being approximately 0.11 m thick as shown in Fig. 5(b)]. Based on a limited number of intrusive inspections, URM cavity infill wall wythes were usually observed to be tied together by 4-mm-diameter Warrington steel wire cavity ties twisted in a figure-eight configuration, which were generally spaced at 4–5 brick lengths horizontally by 4–5 bricks heights vertically in a staggered arrangement. One URM cavity infill wall panel tested in a contemporary building in Hastings in a vertically spanning condition was empirically determined to have a strength out-of-plane less than 60% of the ULS demands (Walsh et al. 2014). The other 30% of the buildings with URM infill wall panels were identified as having solid URM infill panels with two or three contiguous wythes of clay bricks.



(a) (b)

Fig. 5. Unreinforced clay brick masonry (URM) infill wall construction: (a) full-height URM infill panels on an Art Deco building in Ahuriri, Napier, New Zealand; (b) typical URM cavity infill wall construction with plaster façade and single brick removed, exposing the air cavity separating two single wythes of clay brick (images by Kevin Q. Walsh)

Of those Napier Art Deco buildings identified as having either cavity or solid URM infill walls, approximately 63% had at least some infill panels that spanned the full height of the frame, in which case cementious (or, less frequently, lime) mortar joints connected the URM infill panels to the surrounding frame at both top and bottom. In the other 37% of cases, URM infill panels only rose partial-height within any given perimeter frame, truncated most usually by window or door frames spanning the entire length of the frame. The average height of a partial-height masonry infill panel was 1.35 m (i.e., approximately one-third of the mean ground floor frame clear-span height). Measured frame clear-span height and length distributions are charted in Figs. 6(a and b), respectively. The average clear-span heights are 3.94, 3.38, and 3.44 m for stories progressing from the ground to the roof. The average clear-span length is 4.00 m. The dimensions charted in Figs. 6(a and b) also approximate the story heights and horizontal column spacings, respectively, and are merely short the corresponding beam depth (typically 0.35–0.70 m) and column width (typically 0.30–0.50 m), respectively. Asymmetrically placed infill panels, especially on corner buildings, are also capable of causing plan irregularities. However, Dowrick and Rhoades (2000) determined that low-rise RC-framed buildings with asymmetrically placed wall panels rarely suffered more damage in numerous historic New Zealand earthquakes than did their counterparts with symmetrically placed infill walls, and the presence of infill walls likely benefitted several buildings during earthquakes.



(a) (b)

Fig. 6. Distributions of RC frame geometric measurements in Napier's Art Deco building stock: (a) frame clear-span height distribution by story; (b) perimeter frame clear-span length distribution

MATERIAL PROPERTIES

Engineers performing detailed seismic assessments of interwar RC structures in New Zealand and elsewhere can derive more accurate, less conservative results when enhanced knowledge of expected material properties is available. Chapman (1991) provided sampling data of steel reinforcement extracted from New Zealand highway bridges, which is referenced by both NZSEE (2006) and Transit New Zealand (now called the NZ Transport Agency) (TNZ 2004) in regard to expected steel reinforcement material properties. However, because of the lack of historic data regarding steel reinforcement in historic buildings in New Zealand, NZSEE (2006) emphasizes that "whenever practicable, samples of steel [reinforcement] from the structure [being considered for assessment] should be tested." Consequently, Table 1 includes a summary of the results of laboratory tests of material samples extracted where permissible from contemporary buildings in Hawke's Bay. Mean steel reinforcement yield strengths were 304 and 270 MPa for samples extracted from the buildings in Napier and Hastings, respectively, and tested in accordance with ASTM A370-12a (ASTM 2012). NZSEE (2006) and TNZ assessment guidelines serve as references against which to gauge the outcomes of the steel reinforcement material tests, with NZSEE (2006) prescribing for analysis a characteristic mean yield strength of 300 MPa and minimum yield strength of 227 MPa (i.e., ASTM grade 33) for steel reinforcement produced as early as 1930. Alternatively, NZSEE (2006) recommends that the lower characteristic yield strength be multiplied by 1.08 to approximate the yield strength used for assessment (i.e., 245 MPa for ASTM grade 33). TNZ (2004) prescribes for analysis a characteristic minimum yield strength of 210 MPa for steel reinforcement produced up to 1,932 and 250 MPa for the period 1933–1966. The yield strains for the tested specimens shown in Table 1 are typical of modern steel reinforcement, and the ultimate strains measured are high enough such that a member's performance is unlikely to be limited owing to rupture of steel reinforcing bars.

Table 1. Material properties of samples extracted from select Art Deco buildings

Characteristic	# Tests	Test mean	Test standard deviation	Test lower bound	Ref. mean or median	% diff. mean or med.	Ref. "lower bound"	% diff. lower bound
Steel reinforcement samples from a 1929/1931 building in Napier								
Yield strength (MPa)	5	304	19	273	245 - 300	+1% - +19%	210 - 227	+17% - 23%
Yield strain (mm/mm)	3	0.0029	0.001	0.00125	-	-	-	-
Ultimate strength (MPa)	5	425	22	389	-	-	-	-
Strain at ultimate strength (mm/mm)	5	0.1147	0.0235	-	-	-	-	-
Steel reinforcement samples from a 1929 building in Hastings								
Yield strength (MPa)	11	270	41	202	245 - 300	-11% - +9%	210 - 227	-4% - -12%
Yield strain (mm/mm)	11	0.0019	0.0004	0.0012	-	-	-	-
Ultimate strength (MPa)	13	399	50	317	-	-	-	-
Strain at ultimate strength (mm/mm)	11	0.1309	0.0677	-	-	-	-	-
Concrete core samples from a 1929 building in Hastings								
Max compressive strength (MPa)	5	30.3	11.19	11.8	21	+31%	14	-18%
Elastic modulus (GPa)	5	26.5	5.36	-	-	-	-	-

For general purposes, NZSEE (2006) equates anticipated characteristic minimum strength (i.e., 5th percentile) with the specified nominal material strength. Although test material sample sizes as small as three or five samples are not ideal, utilizing the characteristic lower bound strength results in a generally conservative assumption for analysis nonetheless. The steel reinforcement material samples extracted from the building in Napier were slightly stronger than expected, consistent with the findings of Chapman (1991), and the samples extracted from the building in Hastings were slightly weaker than expected. Although these comparative observations may be associated with randomness because of the relatively localized and small selection of samples, the latter finding of steel reinforcement in the Hastings building that is weaker than is recommended by NZSEE (2006) for assessment is notable, and it emphasizes the need for material tests while carrying out a proper detailed seismic assessment for any given building in the region.

Table 1 shows the strength and stiffness of a series of compression tests of concrete cylinder cores [tested in accordance with NZS (1986)] that were 95 mm in diameter and extracted from the same building in Hastings from which steel reinforcement material samples were taken. The mean compressive strength of the concrete cores was 30.3 MPa. In comparison, TNZ (2004) prescribes a specified nominal compressive strength of 14 MPa for concrete produced up to 1,932 and 17 MPa for the period 1933–1940. NZSEE (2006) and Priestley (1996) recommend multiplying the nominal strength by 1.5 (equaling 21 MPa) to estimate the strength assumed for assessment. The characteristic lower bound compressive strength from the test results shown in Table 1 is slightly lower than anticipated owing to the high variance and small sample size of the test results.

COLUMN GEOMETRY AND REINFORCEMENT DETAILING

No design code for concrete structures existed in the United Kingdom before 1934 (Clarke 2009). Furthermore, no building standard existed in New Zealand until 1935 (MacRae et al. 2011), although bylaws directing building construction were approved in the wake of the 1931 Hawke's Bay earthquake (New Zealand Parliament 1931). Although no standards were specified in these bylaws for the detailing of RC structures, detailing standards were specified for the construction of RC bond beams of load-bearing URM walls, which likely informed the detailing of beams and columns in RC buildings. The RC bond beams were required to be at least four brick wythes deep (i.e., approximately 0.45 m) and as wide as the brick wall on which they rested. These bond beams were required to have longitudinal steel reinforcement greater than 0.8% of the concrete cross section with no fewer than four longitudinal bars, with one bar placed in each corner of the cross section. Transverse ties were required to be placed at longitudinal spacings not exceeding 0.30 m, using 6.4-mm-diameter round steel bars. Because of the relative success of the performance of RC buildings during the 1931 Hawke's Bay earthquake (Mitchell 1931; Brodie and Harris 1933; Dowrick 2006) and based on comparisons of buildings constructed before and after the earthquake, RC construction practices likely remained consistent through the interwar time period.

The load and displacement capacity provided by the geometry and the steel reinforcement detailing in the ground floor columns of interwar RC buildings is an especially critical consideration for the assessment of collapse prevention and expected damage concentration (Brodie and Harris 1933; Dowrick 1998). Column geometries and steel reinforcement detailing of ground floor columns were assessed for seven Art Deco buildings (Table 2), where either access was available to undertake intrusive inspections or for which robust building plans were readily available. Six of these buildings were constructed (or reconstructed) in either 1931 or 1932, and the seventh was constructed in 1929 and survived the 1931 earthquake with minimal damage. The structural systems represented by these seven buildings include two-way RC frame (six buildings) and RC frame with RC shear walls (one building). The architects for six of these buildings are among the four most prominent architects shown in Fig. 4(d). Although these seven buildings represent a relatively small sample size, the traits of these buildings (i.e., age of construction, structural system, number of stories above grade, and architect) were appropriately representative of the most common traits found across the larger Art Deco building stock (Figs. 3 and 4).

Table 2. Buildings considered for sampling ground floor column geometry

Building	**Architect**	**Gross footprint (m^2)**	**Structural footprint ratio**	**Estimated structural displacement ductility capacity**
2-storey, RC frame (1923/1932)	JA Louis Hay	448	2.6%	2.0
2-story, RC frame (1932)	EA Williams	860	1.2%	2.1
3-story, RC frame (1932)	JA Louis Hay	84	2.3%	2.4
2-story, RC frame (1932)	JA Louis Hay	312	2.2%	2.2
2-story, RC frame with shear walls (1932)	Finch & Westerholm	304	> 3% (incl. shear walls)	2.1
3-story, RC frame (1929/1931)	HA Westerholm	248	1.7%	2.1
Weighted average for buildings with rectangular columns		**376**	**1.9%**	**2.1**
3-story, RC frame (1929)	Edmond Anscombe	1083	0.6%	1.8
Weighted average for all buildings		**477**	**1.5%**	**2.0**

Information pertaining to the ratio of the sum of RC column and wall cross-sectional areas on the ground floor to the total building footprint area, or structural footprint ratio, for each of the seven sample buildings is provided in Table 2. The average structural footprint ratio of 1.5% shown in Table 2 was weighted by the respective gross footprint areas of the seven buildings. The average structural footprint ratio, and all but one of the individual ratios, exceeds the ratios of 0.6 and 0.9% recommended as minimums by Glogau (1980) for two-story and three-story buildings, respectively, based on a study of the seismic performance of low-rise RC buildings of limited ductility in Japan. In a study specific to low-rise RC building performance in the 1931 Hawke's Bay earthquake, van de Vorstenbosch et al. (2002) determined that open moment-resisting frame systems with structural footprint ratios of approximately 0.4% or greater performed well, and infilled moment-resisting frames performed well with structural footprint ratios as low as 0.3% (excluding the masonry area). It is anticipated that the high structural footprint ratios of many of the Art Deco buildings would limit ULS drift demands imposed on the columns by the DBE. For example, the last building listed in Table 2 has a notably lower structural footprint ratio than the rest and also rises three stories above grade. However, despite these disadvantages, this particular building survived the 1931 earthquake with only superficial damage and is expected to behave relatively rigidly with limited interstory drifts (e.g., <1.3%) as determined by a nonlinear time-history analysis (NLTHA) (Walsh et al. 2013).

Estimated performance characteristics for each of 28 different column types (representing 212 columns in total) identified in the ground floor of the seven buildings are listed in Table 3. Eleven columns (representing only 5% of all RC columns in the seven buildings considered) were not included in Table 3 because of their odd shapes and difficulty of assessing in a manner consistent with the 212 columns that were evaluated. Average values listed at the bottom of Table 3 were weighted by the number of columns of each type and are considered for two groupings of buildings: (1) the six prototypical buildings with rectangular columns and higher structural footprint ratios; and (2) for all seven buildings, including the less prototypical building with circular columns and a relatively low structural footprint ratio. For buildings in which material test results were not available, mean concrete ultimate compressive strength was assumed to be 21 MPa, and the mean steel reinforcement yield strength was assumed to be 245 MPa. Concrete clear cover thickness was assumed to be 40 mm when not called out on the plans. The 212 RC columns identified from among the seven buildings were all identified as having a longitudinal reinforcement cross-sectional area greater than 0.8% of the gross column cross-sectional area. Where it was possible to identify visually or in plan sets, all transverse reinforcement hoops were deemed to have hooks greater than or equal to 135 degrees, or to consist of spiral reinforcement. The other most critical consideration for column ductility is the detailing of transverse reinforcement in terms of spacing (s or s / h in Table 3) and cross-sectional area (ρ_s in Table 3). Over half of the columns assessed (by total number rather than by type) have a transverse reinforcement spacing ratio, s / h , (weighted average of 0.44) complying with the relevant criterion necessary to be classified as likely to fail in flexure (i.e., less than or equal to 0.50) according to ASCE 41-13 (ASCE 2014). Conversely, fewer than half the columns assessed have a transverse reinforcement ratio, ρ_s , (weighted average of 0.18%) complying with the relevant criterion necessary to be classified as likely to fail in flexure (i.e., ρ_s greater than or equal to 0.20%) according to ASCE 41-13. The outstanding factor in the estimation of failure condition according to the criteria of ASCE (2014) is the ratio of the estimated shear force corresponding to the maximum moment capacity of the column and the estimated shear capacity of the column degraded accordingly to account for flexural ductility demand (values for V_p / V_n in Table 3) using the method proposed by Sezen and Moehle (2004). The expected failure condition as determined by the criteria of ASCE (2014) for each column type is listed in the last column of Table 3. Results from tests of similarly detailed columns with smooth reinforcement have shown that the ASCE 41-13 (ASCE 2014) criteria are generally conservative (Ricci et al. 2013).

The estimated curvature ductility capacity (μ_φ) for each column type as listed in Table 3 was determined using the empirically based equations from Priestley (1998). The weighted average curvature ductility capacity for all columns considered is 26.4. This value is nearly equal to the conservative upper limit of 27 used in the current New Zealand design standard (NZS 2006) to develop the prescribed design limit on transverse reinforcement ratio in RC columns for the ductile design criteria. The values of plastic rotation capacity (θ_p) as listed in Table 3 were determined assuming that the plastic hinge length for each column was equal to half the column depth (NZSEE 2006), and the estimated plastic rotation capacities are consistent with measured plastic hinge rotations from tests of similarly detailed columns with smooth, round reinforcement (Ricci et al. 2013).

Table 3. Ground floor column geometry and detailing

Building	# cols. by type	Dimensions, h x b (mm x mm)	(# longitudinal reinf. bars) - diameter (mm)	(# transverse hoops) – diameter (mm) @ spacing (mm)	Hooks (degs)	A_s / A_g	s / h	V_p / V_n	$N / (A_g * f'_c)$	ρ_s	μ_φ	θ_p	δ_s	ASCE 41-13 (ASCE 2014)	
														m-factor	Failure cond.
2-storey, RC frame (1923/1932)	2	533 x 762	(10) - 22.2 mm dia.	(1) - 6.4 mm dia. @ 152	180	0.010	0.29	0.58	0.01	0.0006	39.5	0.050	3.1%	1.42	F-S
	15	457 x 457	(8) - 28.6 mm dia.	(1) - 6.4 mm dia. @ 152	180	0.025	0.33	0.98	0.04	0.0011	18.5	0.023	3.1%	1.53	F-S / S
	4	457 x 533	(8) - 28.6 mm dia.	(1) - 6.4 mm dia. @ 152	180	0.021	0.33	0.86	0.02	0.0009	23.2	0.029	3.1%	1.49	F-S
	3	457 x 457	(4) - 28.6 mm dia.	(1) - 6.4 mm dia. @ 152	180	0.012	0.33	0.70	0.03	0.0011	26.0	0.032	3.2%	1.53	F-S
	1	457 x 1219	(10) - 25.4 mm dia.	(1) - 6.4 mm dia. @ 152	180	0.009	0.33	0.66	0.02	0.0004	30.0	0.038	3.0%	1.30	F-S
	1	457 x 610	(4) - 22.2 mm dia.	(1) - 6.4 mm dia. @ 152	180	0.006	0.33	0.39	0.02	0.0008	48.3	0.061	3.2%	1.47	F-S
	1	457 x 457	(4) - 22.2 mm dia.	(1) - 6.4 mm dia. @ 152	180	0.007	0.33	0.47	0.02	0.0011	40.4	0.051	3.3%	1.53	F-S
2-story, RC frame (1932)	4	406 x 406	(8) - 22.2 mm dia.	(2) - 6.4 mm dia. @ 114	180	0.019	0.28	0.61	0.03	0.0030	36.4	0.046	3.9%	2.13	F / F-S
	4	406 x 406	(6) - 25.4 mm dia.	(1) - 6.4 mm dia. @ 114	180	0.018	0.28	0.91	0.03	0.0017	23.8	0.030	3.3%	1.66	F-S
	2	610 x 457	(6) - 22.2 mm dia.	(1.5) - 6.4 mm dia. @ 114	180	0.008	0.25	0.46	0.01	0.0016	54.7	0.070	3.5%	1.64	F-S
	15	330 x 381	(6) - 22.2 mm dia.	(1) - 6.4 mm dia. @ 114	180	0.019	0.35	0.75	0.06	0.0019	24.9	0.031	3.3%	1.71	F-S
	2	406 x 406	(8) - 22.2 mm dia.	(2) - 6.4 mm dia. @ 114	180	0.019	0.28	0.60	0.01	0.0030	47.7	0.061	3.9%	2.13	F / F-S
	4	457 x 457	(10) - 25.4 mm dia.	(2) - 6.4 mm dia. @ 114	180	0.024	0.25	0.93	0.01	0.0026	32.8	0.041	3.6%	1.86	F-S
	4	394 x 711	(6) - 22.2 mm dia.	(1) - 6.4 mm dia. @ 114	180	0.008	0.29	0.53	0.01	0.0009	40.1	0.051	3.2%	1.49	F-S
	4	711 x 711	(12) - 22.2 mm dia.	(2) - 6.4 mm dia. @ 114	180	0.009	0.16	0.48	0.02	0.0015	50.8	0.065	3.4%	1.62	F-S
	1	330 x 330	(4) - 19.1 mm dia.	(1) - 6.4 mm dia. @ 114	180	0.010	0.35	0.60	0.03	0.0023	36.4	0.046	3.6%	2.04	F / F-S
	1	381 x 381	(8) - 19.1 mm dia.	(2) - 6.4 mm dia. @ 114	180	0.016	0.30	0.55	0.03	0.0032	44.3	0.056	4.0%	2.15	F
	2	305 x 305	(4) - 19.1 mm dia.	(1) - 6.4 mm dia. @ 114	180	0.012	0.38	0.48	0.03	0.0025	38.1	0.048	3.8%	2.06	F
	2	457 x 457	(10) - 19.1 mm dia.	(2) - 6.4 mm dia. @ 114	180	0.014	0.25	0.59	0.02	0.0026	44.9	0.057	3.7%	2.08	F / F-S
3-story, RC frame (1932)	12	356 x 356	(4) - 25.4 mm dia.	(1) - 9.5 mm dia. @ 203	135	0.016	0.57	0.75	0.05	0.0025	27.9	0.035	3.6%	1.84	F-S
2-story, RC frame (1932)	8	406 x 457	(8) - 19.1 mm dia.	(1) - 9.5 mm dia. @ 203	135	0.012	0.50	0.65	0.05	0.0019	31.3	0.039	3.4%	1.71	F-S
	9	406 x 457	(6) - 19.1 mm dia.	(1) - 9.5 mm dia. @ 203	135	0.009	0.50	0.56	0.02	0.0019	40.7	0.052	3.5%	2.00	F / F-S
	10	432 x 737	(4) - 19.1 mm dia.	(1) - 9.5 mm dia. @ 203	135	0.004	0.47	0.31	0.01	0.0011	71.2	0.091	3.3%	1.53	F-S
	2	457 x 457	(8) - 19.1 mm dia.	(1) - 9.5 mm dia. @ 203	135	0.011	0.44	0.61	0.04	0.0019	34.9	0.044	3.4%	2.00	F / F-S
2-story, RC frame with shear walls (1932)	12	254 x 254	(6) - 19.1 mm dia.	(1) - 6.4 mm dia. @ 152	135/18	0.027	0.60	0.91	0.10	0.0022	15.8	0.019	3.3%	1.77	F-S
	12	254 x 305	(6) - 19.1 mm dia.	(1) - 6.4 mm dia. @ 229	135/18	0.022	0.90	0.93	0.04	0.0012	17.3	0.021	3.1%	1.55	F-S
3-story, RC frame (1929/1931)	33	356 x 356	(4) - 25.4 mm dia.	(1) - 6.4 mm dia. @ 254	?	0.016	0.71	0.90	0.02	0.0009	19.0	0.027	3.1%	1.49	F-S
Weighted average for rect. columns		**418 x 418**	-		**> 135**	**0.016**	**0.50**	**0.76**	**0.04**	**0.0016**	**29.2**	**0.037**	**3.3%**	**1.67**	**F-S**
3-story, RC frame (1929)	42	447 (diam.)	(8) - 28.6 mm dia.	(1) - 6.4 mm dia. @ 90	spiral	0.033	0.20	1.52	0.17	0.0025	15.2	0.017	2.9%	1.00	S
Weighted average for all		**414 x 414**	**-**		**> 135**	**0.020**	**0.44**	**0.91**	**0.06**	**0.0018**	**26.4**	**0.033**	**3.2%**	**1.53**	**F-S**

A_g = gross cross-sectional area of column; A_s = total cross-sectional area of longitudinal reinforcement; A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) in one direction within spacing s; h = minimum cross-sectional dimension of column; F = flexure condition expected to control; F-S = flexure-shear condition expected to control; f'_c = expected cylinder compressive strength of concrete (NZSEE 2006); *m-factor* = component demand modification factor to account for expected ductility (ASCE 2014); N = axial compressive force on column accounting for seismic gravity loads [including reduced live/imposed loads as per NZSEE (2006)]; s = center-to-center spacing of transverse reinforcement measured along longitudinal axis of column (as provided or as required); S = shear condition expected to control; V_n = shear capacity of column degraded accordingly for flexural ductility demand (Sezen and Moehle 2004); V_p = shear force corresponding to the plastic moment capacity of the column; δ_s = estimated lateral drift at shear failure (not necessarily equal to the drift at axial failure) (Elwood and Moehle 2006); θ_p = estimated max. plastic hinge rotation, assuming hinge length of $h/2$; μ_φ = estimated curvature ductility ratio assuming flexure about weak axis (NZSEE 2006); $\rho_s = A_{sh} / (b * s)$ = minimum ratio of transverse reinforcement in either orthogonal direction.

The expected drift capacity for each column at shear failure (δ_s) as listed in Table 3 was determined by applying the method proposed by Elwood and Moehle (2006). The estimated column drift capacities were considered in the determination of the estimated structural displacement ductility capacities for the seven considered buildings listed in Table 2. These displacement ductility capacities were estimated based on a conservative assumption of structural interstory yield drift occurring at 1.5% (Priestley 1998). The estimated displacement ductility capacity was chosen as the lower of the value derived from the approximation methods proposed by Priestley (1996) (assuming a column-sway soft-story mechanism) and the ratio of the smallest drift at shear failure for any ground floor column in the considered building to the assumed yield drift of 1.5%, resulting in estimated weighted average displacement ductility capacities of 2.1 for the six buildings with rectangular columns and 2.0 for all seven buildings (Table 2). However, these estimated displacement ductility capacities are tempered by the slightly lower *m-factors* determined from ASCE 41-13 (ASCE 2014) criteria, which represent component demand modification factor[s] to account for expected ductility when performing linear seismic assessments. At low fundamental periods (e.g., 0.35 s, which approximates the fundamental period for most Napier Art Deco buildings), the structural displacement ductility capacity, μ , in NZS (2004) is approximately equal to the seismic load reduction factor, k_u / S_p , in NZS 1170.5:2004, which is effectively equivalent to the *m-factor* in ASCE 41-13 (although the *m-factor* is used to enhance capacity rather than reduce demand and is directly applicable to a specific structural component rather than the structural system). Hence, the values of the *m-factors* listed in Table 2 can be assumed to represent limits on the structural displacement ductility capacity assumed for the buildings. The weighted average *m-factor* from this investigation for the columns of the six buildings with rectangular columns is 1.67. Considering the conservatism inherent in the ASCE 41-13 (ASCE 2014) criteria as well as the higher ductility capacities determined through alternative methods, it is recommended that New Zealand engineers performing seismic assessments of these interwar RC buildings should scale the applied earthquake demands assuming a level of structural displacement ductility capacity equal to at least 1.75 unless a critical weakness is identified in the RC frame geometry or detailing. Many Art Deco buildings are expected to have structural displacement ductility capacities exceeding 2.0. For reference, NZSEE (2006) recommends that RC buildings comprised of columns of limited ductility are expected to achieve a structural displacement ductility capacity between 2.0 and 3.0. In comparison, RC buildings comprised of columns designed to be fully ductile in accordance with the modern design standard (NZS 2006) are expected to achieve a structural displacement ductility capacity of as much as 6.0, whereas nonductile or nominally ductile RC frames are expected to achieve a structural displacement ductility capacity less than 1.25 (NZSEE 2006).

Notice the low anticipated axial loads [values for $N / (A_g * f'_c)$ in Table 3] as a result of the relatively close column spacing in the low-rise buildings considered. Column confinement is less likely to control drift capacity when axial loads are relatively low. Hence, potentially limiting factors not considered in the column analysis summarized in Table 3 are the effects of bidirectional loading on columns with limited ductility (Boys et al. 2008), premature longitudinal bar buckling, and bond slip of anchorage or lap splice connections (NZSEE 2006). The latter consideration is especially noteworthy because all reinforcement in the columns considered in Table 3 was identified as being comprised of smooth, round bar. Various other studies into RC columns with similar detailing have found that round longitudinal bars with ineffective anchorage or development length can induce failure at relatively low loads, but that properly spliced or anchored round longitudinal bars are likely to effectuate higher column deformability than deformed bars (Ricci et al. 2013) and induce rocking mechanisms at the base of the columns (Arani et al. 2013). Where identified on plans, longitudinal reinforcement in the columns listed in Table 3 was anchored using 180-degree hooks.

FOUNDATIONS

One further observation was made regarding the seven buildings listed in Table 2. All seven of these buildings were identified as having shallow foundations, typically comprising of spread footings beneath the columns and grade beams or foundation tie beams spanning between spread footings, often with thick concrete slabs (or raft foundations) in the basements. These traits are consistent with the observations of Brodie and Harris (1933) for RC buildings in Hawke's Bay, who noted that these foundation types performed satisfactorily in the 1931 earthquake. Dowrick (1998) observed that deep foundations for RC buildings were not in use in Hawke's Bay during the interwar period.

OBSERVED DAMAGE TO SIMILAR STRUCTURES IN THE CANTERBURY EARTHQUAKES AND ELSEWHERE

The 2010–2011 Canterbury, New Zealand, earthquakes serve as the most recent empirical precedent for a forensic investigation of Napier's Art Deco buildings. Ten buildings that were constructed during the interwar period, constructed of RC frames, and between two and four stories in height were identified in databases and literature describing building damage following the February 22, 2011, Christchurch earthquake [Canterbury Earthquakes Recovery Authority (CERA) 2012; Pampanin et al. 2012]. Observed damage was typically limited to the exterior of the buildings because inspectors could not safely enter most of the premises. Shear cracking was observed in several exterior columns (some severe). Several masonry infill walls were also heavily cracked, and an exterior URM infill wall in one of the considered buildings even partially collapsed out-of-plane. However, none of the ten interwar RC buildings considered experienced the collapse of primary structural components during the February 22, 2011, earthquake, despite the unusually high earthquake intensity (Fig. 7, with a comparison of response spectra from records in the 2011 Christchurch earthquake to the DBE response spectrum for Napier). Furthermore, the contemporary Christchurch buildings considered had already been weakened by the preceding September 4, 2010, Darfield earthquake. Hence, the observed satisfactory life-safety performance of these interwar RC buildings in Christchurch should increase engineers' confidence regarding the probable satisfactory life-safety performance of Napier's Art Deco buildings under DBE loading.

The fatal collapses of two relatively modern RC buildings in Christchurch (Cooper et al. 2012), constructed in 1966 and 1986, brought greater attention to deficiencies in newer-type RC construction. Even before the 2011 Christchurch earthquake, however, RC buildings of the more modern era were of generally greater concern to engineers and emergency managers in New Zealand. NZSEE (2006) noted that

> *Reinforced concrete buildings from the 1940s and the 1950s are typically low-rise with regular and substantial wall elements. Many of these structures would be capable of close to an elastic level of response, with local detailing exceptions. Reinforced concrete buildings from the 1960s and early 1970s are, however, generally taller, less generously proportioned, with less redundancy and greater irregularity often in evidence in frame structures.*

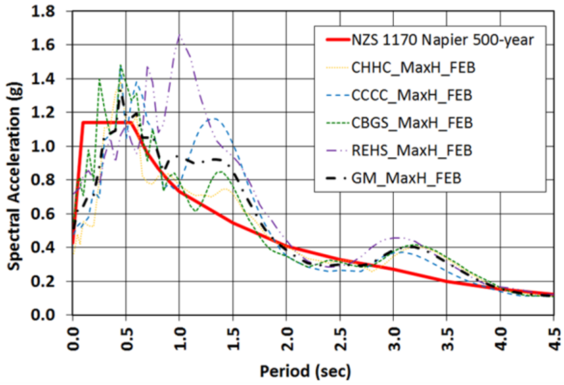


Fig. 7. NLTHA target response spectrum for Napier assuming deep soils [derived from NZS 1170.5:2004 (NZS 2004)], maximum horizontal ground motions recorded at four sites near the Christchurch city center during the February 22, 2011 earthquake, and the geometric mean (GM) of those four ground motions (data from GeoNet 2013)

Observations of RC building damage from major earthquakes around the world are also consistent with the NZSEE observations. Citing data accrued by the Architectural Institute of Japan [summarized by Otani (1999)] from the sites of four of the more significant earthquakes of the past thirty years (1985 Mexico, 1990 Luzon, 1992 Erzincan, and 1995 Kobe), Wu et al. (2009) noted that "the probability of structural collapse in older-type concrete buildings was relatively low (1.9–6.6%) even in such damaging earthquake events," where it appears that Wu et al. (2009) may have been referring to older-type buildings as having been constructed prior to approximately 1980. Furthermore, the Otani (1999) data illustrate the relatively low risk of collapse associated with RC buildings with no more than three stories, which is the maximum height associated with Napier's Art Deco building stock. The 1985 Mexico earthquake ground-motion records are one of the seven recommended sources of frequency content when performing an NLTHA in Hawke's Bay (Oyarzo-Vera et al. 2012).

SUMMARY AND CONCLUSIONS

Typological and geometric investigations were performed on sample groups of Art Deco buildings in Napier, New Zealand, constructed during the interwar period, with the primary objectives being to provide initial guidance on the seismic risk of these buildings to interested parties and to help facilitate more accurate detailed assessments in the future. Specific conclusions derived from this investigation are as follows:

- The Art Deco building stock is relatively homogeneous, and the homogeneity of the building stock can be leveraged so as to make findings from empirical studies (e.g., performance of similar buildings in past earthquakes) as well as exemplar detailed assessments useful to a large number of buildings within the portfolio.
- The prototypical Art Deco building was constructed in the first half of the 1930s, has two stories, is abutted on either side by its neighbors in a row-style footprint, was designed by one of only five architecture firms, and is comprised of a two-way RC space frame with clay brick masonry infill in at least some portion of the building.
- The prototypical Art Deco building with rectangular columns and two-way framing has a gross footprint of approximately 380 m^2 and a structural footprint ratio of approximately

1.9%, the latter of which is a relatively high value that can be associated with successful performance in historic earthquakes in New Zealand and internationally.

- The prototypical RC rectangular column of an Art Deco building is rectangular in cross section with dimensions of approximately 420 x 420 mm and has transverse reinforcement hooked inward toward the confined core at angles equal to or greater than 135 degrees. This transverse reinforcement is spaced longitudinally at about half of the smallest dimension of the column and, coupled with low axial loads, is expected to provide sufficient deformation capacity to facilitate a limited ductile behavior in the column when subjected to lateral loads. The expected failure condition in the prototypical Art Deco RC rectangular column is expected to be flexure preceding shear cracking, as was observed in similar buildings damaged by the 2011 Christchurch earthquake.
- The observed performance of similar low-rise, ostensibly brittle RC buildings in the 1931 Hawke's Bay earthquake, the 2011 Christchurch earthquake, and other historic earthquakes, both in New Zealand and elsewhere, lends credence to the conclusion that these buildings are generally underrated in simple, force-based seismic assessments.

Recommendations derived from this study for structural engineers performing detailed seismic assessments on buildings similar to those discussed in this paper are as follows:

- Common potential vulnerabilities identified among the Art Deco buildings that should be closely considered are nonstructural falling hazards (namely slender, inadequately reinforced parapets and URM infill walls, especially atop taller buildings), pounding potential with neighboring buildings with offset building or story heights, splice or anchorage failure of smooth reinforcement bars, premature buckling of longitudinal reinforcement in columns, and column shear strength degradation at high ductility demands.
- Assumed material strengths for use in assessing these buildings should generally be approximately 245–270 MPa for the tensile yielding of steel reinforcement and 14–21 MPa for the ultimate compressive strength of concrete.
- Infill walls may have contributed greatly to the successful performance of similar buildings in previous earthquakes and should be included in detailed seismic assessments as being generally beneficial elements in-plane (provided they do not create short columns when only partial height), even for buildings where infill walls appears to create asymmetries in plan.
- Assessment efforts seeking accuracy and the avoidance of excessive conservativeness should be focused on leveraging the inherent stiffness and redundancy of the complete structures limiting the estimated interstory drift demands instead of, as is traditionally done, being focused on increasing the estimated strength capacity of structural elements. Practically, this approach requires that engineers utilize system-oriented assessment techniques such as modal response spectrum analyses or, preferably, displacement-based assessment techniques such as nonlinear procedures (where the nonlinear drift demands and capacities of columns and other frame elements are considered).
- New Zealand engineers performing seismic assessments of Art Deco RC buildings should reduce the applied earthquake demands assuming a level of structural displacement ductility capacity of at least 1.75 (to be further considered in future research efforts) unless a critical weakness is identified in the RC frame geometry or detailing. Many Art Deco buildings are expected to have structural displacement ductility capacities exceeding 2.0.

- Engineers should consider the strong empirical evidence supporting the historic success of these types of buildings when interpreting the results of detailed assessments.

Building officials and insurers are encouraged to request more sophisticated analyses before enforcing ordinances, policies, or premiums that could lead to vacancies and building demolitions. Further efforts to expand on these findings will provide greater clarity on the seismic risks associated with the Art Deco building stock in Napier. In future efforts, the researchers will perform and report on the results of a detailed assessment of a representative Art Deco building. The results of this latter study could also be incorporated into fragility curve functions particular to Hawke's Bay's Art Deco building stock to enhance the accuracy of New Zealand's seismic hazard models and either validate or improve upon previous work done internationally in this realm (e.g., Shoraka et al. 2012). Further study on the deformation capacity of columns with smooth, round longitudinal reinforcement (including in situ testing, if possible) would help refine the estimation of ductility capacities for these buildings.

ACKNOWLEDGMENTS
The authors would like to thank those associated with the Napier Art Deco Trust (http://www.artdeconapier.com) for providing the authors with access to the case-study buildings and to building plans. Consultants at EQ Struc, Ltd. (http://www.eqstruc.co.nz) and Strata Group (http://www.stratagroup.net.nz) provided the authors with some of the material test specimens and typological information. Win Clark of NZSEE (http://www.nzsee.org.nz) provided guidance and review of preliminary assessment findings. W.Y. Kam and Stefano Pampanin of the University of Canterbury (http://www.civil.canterbury.ac.nz) and Gareth Wright and Philip Barrett of the Christchurch City Council (http://www.ccc.govt.nz) provided the authors with information pertaining to the identification of and documented damage to interwar RC buildings in the 2011 Christchurch earthquake.

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