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EVALUATION OF FAILURE MODES OF R.C. BUILDINGS

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ABSTRACT

Recent earthquakes have indicated that the majority of reinforced concrete building failures were observed in structures that were designed to various versions of earlier design codes. Given that this identification applies to the majority of the building stock in the greatest part of the world today, application of detailed assessment procedures for seismic evaluation of every single existing structure appears unfeasible from the sheer volume of the required work. In this paper, application of a rapid, yet efficient methodology for the evaluation of failure modes of lightly reinforced substandard buildings is presented. The method determines the limiting shear resistance of the structure as the least value supported by the columns' pure flexural, degraded shear, anchorage or lap-splice and joint shear resistance mechanisms. For application of the methodology, only knowledge of the basic geometric and material properties of the building is required. For confirmation, the methodology is applied to two R.C. buildings that failed during the 1999 Athens earthquake. Results indicated that both buildings failed in a brittle manner due to anchorage failure of column longitudinal reinforcement in the joints' regions.

Introduction

Systematic seismic assessment of reinforced concrete buildings designed to various versions of earlier design codes is imperative in countries with a high seismicity, since recent strong earthquakes have underscored their vulnerability. The large number of substandard, lightly reinforced existing buildings renders the massive use of detailed seismic analyses a very demanding work-intensive task, which requires a large number of specially trained engineers. In this paper, a simple yet reliable method for evaluation of failure modes of such "non-conforming" R.C. buildings is presented, that only requires the knowledge of the structural system configuration and its material properties. The method is ideal for rapid preliminary seismic assessment and it can be shown to be a useful diagnostic tool for identifying the

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prevailing mechanism of building failure.

Methodology for Rapid Evaluation of Failure Modes

Reinforced concrete buildings constructed prior to the introduction of capacity design principles and modern detailing practices have structural systems that are characterized by small section columns, relatively stiff beams, inadequately confined joints and insufficient anchorage of longitudinal and transverse reinforcement. For this class of buildings, a rapid evaluation for the potential mode of failure could be focused on the calculation of the limiting strength of building columns, since post-earthquake reconnaissance reports illustrate column failure as the primary cause of building collapse, being related with the loss of gravity load-carrying capacity.

For any individual column, failure is anticipated in the weakest link that develops in its load-carrying system. Thus, $V^c_{u,lim}$ which represents the limiting shear resistance developing in a column subjected to cyclic loading, is the least strength of one of the following resistance mechanisms: (a) the flexural mechanism, $V^c_{u,flex}$, (b) the degraded shear mechanism, $V^c_{u,shear}$, (c) the degraded bar anchorage mechanism, $V^c_{u,anch}$ and (d) the column shear associated with failure of the joints in the column ends, $V^c_{u,joint}\cdot d_b/L_s$, where d_b is the depth of the beam cross section and L_s is the column shear span length (fib Bulletin 2003, Chapter 4) as illustrated by:

$$V^c_{u,lim} = min\left\{V^c_{u,flex}, V^c_{u,shear}, V^c_{u,anch}, V^c_{u,buckl}, V^c_{u,joint}\cdot\left(d_b / L_s\right)\right\} \tag{1}$$

For columns belonging to the structural system of R.C. structures characterized as “non-conforming” according to modern standards (FEMA 356 2000), shear resistance for each of the previously mentioned mechanisms can be calculated from the relationships presented in the following paragraphs.

Flexural Strength

The ideal flexural strength under cyclic loading is meaningful only if it may be safely assumed that it is supported by all other mechanisms of behavior (i.e., if $V^c_{flex} < \{V^c_{shear}, V^c_{anch}, V^u_{joint}\cdot d_b/L_s\}$). In the case of uni-axial bending of concrete members, the flexural yield moment that may be sustained after cyclic reversal of load, M^c_{y0}, is reduced compared to the corresponding moment in a monotonic loading history, M^m_{y0} (Thom 1983). For columns with equally distributed reinforcement, the expression for calculation of cyclic yield moment is presented in Table 1. The uni-axial flexural ultimate moment, M^c_{u0}, can be calculated in the same manner, by replacing the longitudinal reinforcement yielding strength with the corresponding ultimate strength. For columns subjected to bi-axial bending, the ultimate cyclic moment resistance in the i^{th} direction of the two principal directions of the section, $M^c_{u,i}$, can be calculated by reducing the sections' uni-axial ultimate cyclic resistance in the corresponding direction, $M^c_{u0,i}$, by 30%, as presented in Table 1. The column ultimate cyclic shear resistance equals to the quotient of the ultimate cyclic moment by the column shear span, taken here as half the column height (whereby it is assumed that the point of contraflexure of a laterally swaying column is at its midheight).

Shear Strength

Various models have been proposed to establish the shear strength of reinforced concrete as a function of deformation. A common working hypothesis is that the shear strength of cracked reinforced concrete comprises a primary contribution of the web reinforcement, V_w, (the tension ties of the Ritter-Moersch truss analogy) and secondary contributions of the concrete web, V_c . The contribution of the web reinforcement, V_w, is nonzero only if ties are spaced close enough to secure that any diagonal crack (taken for simplicity at an angle of 45º with respect to the longitudinal axis of the column) is crossed by at least one stirrup, as illustrated in Fig. 1. Calculation of V_w and V_c terms is presented in Table 1. To consider the finding of recent tests, where it has become evident that shear strength of reinforced concrete degrades faster with cyclic load for higher ratios of shear demand to shear supply, a limit of 60% to the calculated V^c_{shear} is proposed (Syntzirma and Pantazopoulou 2007).

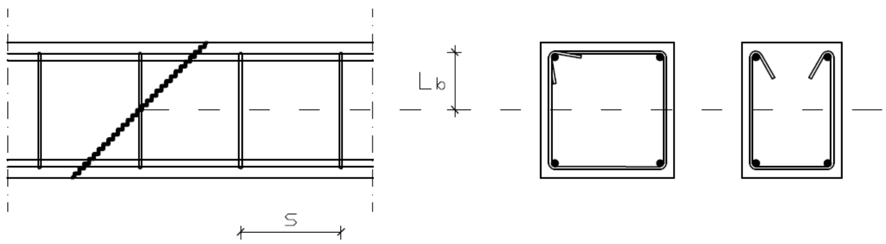


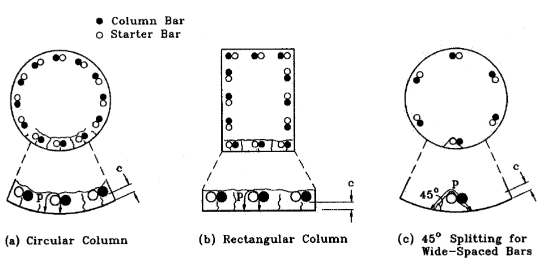
Figure 1. Anchorage length of stirrup, L_b, crossed by the diagonal crack of an angle of 45º with respect to the longitudinal axis of the member.

Anchorage and Lap-Splice Strength

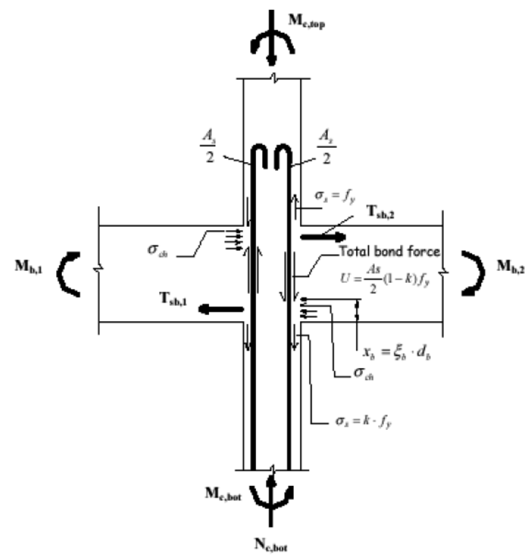
Premature failure of a lap-splice or anchorage effectively limits the force developed in the reinforcing bar to a value lower than its yield strength. In buildings constructed prior to the early 1980's, common bond-related problems are owing to: (a) the practice of splicing the main column reinforcement just above the base of each floor (i.e., within the columns' critical zone), but without special provisions for confinement through stirrups, (b) the use of smooth reinforcement where anchorage capacity depends on frictional mechanisms mobilized along the anchored length and (c) the use of short embedment or lap lengths.

The force that a lap-splice of length L_b at the bottom of a column may develop, is equal to the total frictional force that develops on the bar lateral surface within the length L_b (Priestley et al. 1996). The development capacity of each lapped longitudinal bar of diameter $D_{b,l}$ is obtained from the maximum clamping force that stirrups may provide and the tensile resistance of the concrete cover, f_{ct}, developing over a crack path along the bar cover of length $p = 2\cdot\sqrt{2}\cdot(c + D_{b,l})$ (where c is the concrete cover), as illustrated in Fig. 2(a). Thus, the maximum developed strain of a lapped longitudinal bar at the bottom of a column can be calculated by the first expression of those listed under "anchorage and lap-splice strength" of Table 1. In this equation, the term that accounts for tensile resistance of the concrete cover, f_{ct}, is to be ignored if the maximum axial compression strain exceeds 0.002.

In the case that smooth longitudinal reinforcement is anchored through the beam-column joint in the column above, taking into account the frictional force concept to describe the mechanics of bond, the force that may be supported by the anchorage may be evaluated from the compressive resultant in the compression zone of the adjacent beam cross-section (Calvi et al. 2002). Here, the problem refers to anchorage arrangements as depicted in Fig. 2(b). Whereas ideal anchorage conditions are secured by the hook in the bar segment above the joint, at the lower level compressed longitudinal column bars are actually in tension due to slip in the joint. Transverse compression to the column bars occurs within the joint over a segment $\xi{\cdot}d_b$ that corresponds to the depth of compression zone of the adjacent beam (where ξ is the normalized depth of compression zone and d_b is the depth of the beam cross section). Hence, the maximum frictional force that may develop along the column longitudinal bar within the joint height equals to $N_f{=}\mu{\cdot}\sigma_c{\cdot}\xi{\cdot}d_b{\cdot}\pi{\cdot}D_{b,l}$, where μ is the coefficient of friction and σ_c is the average concrete compressive stress within the compression depth of the beam cross section. To yield the bar by frictional support only, N_f must exceed $F_y = f_y{\cdot}\pi{\cdot}\ D_{b,l}^{\ 2}/4$. Thus, in general N_f is a fraction of F_y, $N_f = \beta{\cdot}F_y$. The corresponding bar force bellow the joint is $F_y - N_f = f_s{\cdot}\pi{\cdot}\ D_{b,l}^{\ 2}$ and the residual bar stress of each of the longitudinal reinforcement anchored at the top of the column is given by the second expression that appears in Table 1. The total anchorage or lap-splice strength of the column can be calculated in a same manner as the calculation of the column strength in pure flexure, after replacement of f_u with f_s.



(a) Development of crack path at lap-slice regions (Priestley et al. 1996)



(b) Column bars anchored through the joint, terminated in hooks (Calvi et al. 2002)

Figure 2. Mechanisms developing at lap-splices and anchorage zones of columns calcified as non-conforming.

Table 1. Expressions utilized in the proposed methodology for estimating flexural and shear strength, anchorage and lap-splice strength, as well as joint shear strength.

Strength	Expressions
Flexural	Uniaxial bending: $M_{y0}^{c} = A_{s1} \cdot f_y \cdot (d - d_2) + N \cdot (d - h/2)$ $M_{u0}^{c} = A_{s1} \cdot f_u \cdot (d - d_2) + N \cdot (d - h/2)$ Biaxial bending: $M_{u,i}^{c} = 0.7 \cdot M_{u0,i}^{c}$, where $i = x$ or y
Shear	$V_{shear} = V_w + V_c$, $V_w = A_{st} \cdot \sum_{i=1}^{n} f_{st,i}$; $f_{st,i} = f_{y,st} L_{b,i} / (a \cdot L_b) \leq f_{y,st}$; α = 0.7 $V_c = \begin{cases} 500 \cdot \sqrt{f_c} \cdot \left(\frac{d}{L_s} \cdot \sqrt{1 + \frac{N}{500 \cdot \sqrt{f_c} \cdot A_g}} \right) \cdot A_g & \text{if} \quad v \geq (\rho_{s1} - \rho_{s2}) \cdot f_y / f_c \\ 0 & \text{if} \quad v < (\rho_{s1} - \rho_{s2}) \cdot f_y / f_c \end{cases}$ where: f_c in MPa ; N in kN and positive for compression ; $\rho_{s1} = A_{s1}/A_g$; $\rho_{s2} = A_{s2}/A_g$ $V_{u,shear}^{c} = 0.6 \cdot (V_w + V_c)$
Anchorage and lap-splice	$f_s = (1.4 \cdot A_{tr} \cdot f_{y,st} \cdot n_{st} / n_b + p \cdot f_{ct} \cdot L_b / \alpha) / (\pi \cdot D_{b,l}^2 / 4)$; α = 0.7 $f_s = \pi \cdot D_{b,l}^2 \cdot f_y \cdot (1 - \beta)$ where $\beta = 0.0357 \cdot d_b / D_{b,l}$, $0 \leq \beta \leq 1$ Calculation of limiting moment and shear strength as in pure flexure, with f_s instead of f_u
Joint shear	Interior Joint: $V_{joint,i}^{c} = f_c^{2/3} \cdot d_{c,i} \cdot (b_{b,j} + b_{c,j}) / 2$ Exterior Joint: $V_{joint,i}^{c} = 0.75 \cdot f_c^{2/3} \cdot d_{c,i} \cdot (b_{b,j} + b_{c,j}) / 2$ where, i, j are the two principal directions

A_g: gross area of the member; A_{s1}, A_{s2}: area of tension and compression reinforcement, respectively; A_{st}: area of stirrup leg; A_{tr}: total area of stirrup legs along one direction of restraint; b_b, b_c: beam and column width, respectively; d, d_2: distance from the tension and compression fiber to the centroid of the tension and compression reinforcement, respectively; d_c, d_b: column and beam depth, respectively; f_c: concrete compressive strength; f_y, f_u: steel yield and ultimate stress, respectively; $f_{y,st}$: steel yield stress of stirrups; h: cross section height; L_b: anchorage or lap length; L_s: shear span length; N: compression force acting in the section from G+0.3·Q load combination (only when exists, with compression as a positive value); $v(=N/A_g \cdot f_c)$: axial load ratio; n_b: total number of bars restrained by a total of n_{st} stirrups

Joint Shear Strength

Preservation of gravity load carrying capacity and lateral load strength in R.C. frame structures under earthquake action is linked to the integrity of the beam-column joints, since these elements are part of both the vertical and horizontal load path. Transfer of forces (shear, moment and axial loads) through the joints is necessary for the development of framing action. Buildings constructed according with old code provisions where neither shear nor bond stress demand were regulated through capacity checks are frequently reported in post-earthquake reconnaissance studies to have experienced joint failures; apart from being very brittle, such failures are a common cause for excessive flexibility of the overall frame and a consequent loss of vertical load carrying capacity (Lehman 2002). To calculate the shear strength of old-type R.C. joints, recommendations of Eurocode 8 (1994) can be utilized. According to these recommendations, the maximum shear strength that a joint can sustain is given by the expression of Table 1.

Methodology Application

The proposed methodology for evaluating the limiting shear resistance of columns of "non-conforming" R.C. buildings is applied to two R.C. buildings that collapsed during the 1999 strong ground motion of Athens. Both buildings were located in the northern region of Athens, were the ground motion possessed "near-field" characteristics. The methodology is applied to the first storey columns of the two buildings, in order to calculate the limiting base shear that the buildings could sustain upon failure, but also to test the ability of the method to identify the high seismic vulnerability of such structures.

Building A was a two-storey fully symmetric in plan industrial building, with external plan dimensions of 38.00 m by 26.00 m (Table 1). The first and the second storey heights were 5.40 m and 5.30 m respectively. Building A was connected with two wing buildings along the two smaller sides. The structural system was formed as an orthogonal grid of columns, beams and slabs, according to typical construction practice of R.C. frame structures in Southern Europe. Slab thickness was 0.15 m. All perimeter beams cross sections were 0.70 m (height) by 0.30 m (width), 0.70×0.45 m for beams spanning between columns and 0.70×0.25 m for the secondary beams. During the earthquake the building collapsed without any horizontal dislocations of its structural elements, while the two adjacent buildings were intact. From tests of core samples, the mean value of concrete compressive strength was determined as 18.7 MPa, whereas steel yielding and ultimate stress was found to be for the longitudinal reinforcement 431.5 MPa and 512.0 MPa, respectively, and for the stirrups 402.0 and 553.0 MPa, respectively. Column stirrups were smooth, rectangular ties, approximately categorized as Ø8/300 mm.

Building B was also an industrial building, having a 37.60 m by 22.80 m orthogonal plan (Table 2). The building had two basements and four storeys, each 2.85 m high. The structural system comprised a grid of columns which were connected only in the buildings' perimeter with beams having a section height of 0.60 m and 0.20 m web width. In the centre of the typical floor plan, columns supported a flat-plate Zoellner system, having a thickness of 0.22 m. During the earthquake the building collapsed, except of the stairwell in the corner of the plan. After tests conducted on material samples , the concrete was found to have a mean compressive strength of 20.0 MPa, while longitudinal reinforcement and stirrups were found to have smooth surface and were classified as S400 (f_y = 400 MPa) and S220 (f_y = 220 MPa) respectively. Column transverse reinforcement comprised Ø6/300 mm rectangular, smooth stirrups.

Typical floor plan drawings and details of the column geometry and reinforcement, for the two buildings are presented in Table 2. Tables 3 and 4 illustrate the limiting strengths of the first floor columns for buildings A and B, respectively, with respect to the X and Y plan directions. Also presented are the column axial loads, calculated for a 100% of the dead and a 30% of the live load of the buildings.

Table 2. Plan and column configuration of the two R.C. buildings used in the methodology application.

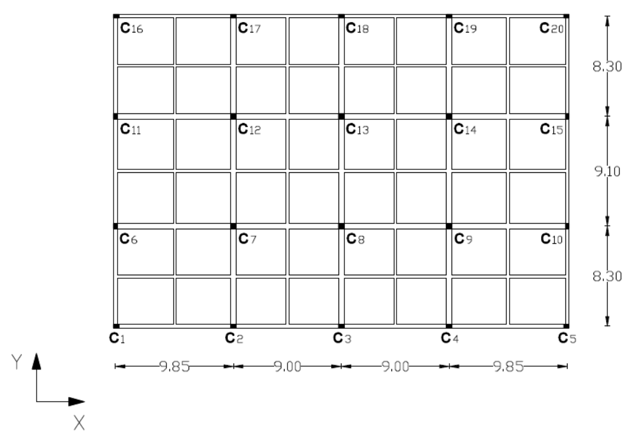
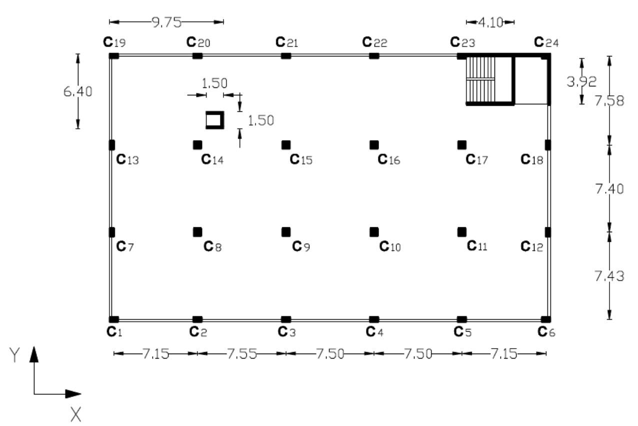
<table>
<tr><th colspan="4">Building A</th></tr>
<tr><th>Typical Plan</th><th colspan="3">Column Details</th></tr>
<tr><td rowspan="5">
</td><th>Column</th><th>Dimensions (mm)</th><th>Reinforcement (mm)</th></tr>
<tr><td>C_{1}, C_{5}, C_{16}, C_{20}</td><td>450 / 300</td><td>4 Ø20</td></tr>
<tr><td>C_{2}, C_{3}, C_{4}, C_{17}, C_{18}, C_{19}</td><td>450 / 300</td><td>8 Ø20</td></tr>
<tr><td>C_{6}, C_{10}, C_{11}, C_{15}</td><td>300 / 450</td><td>8 Ø20</td></tr>
<tr><td>C_{7}, C_{8}, C_{9}, C_{12}, C_{13}, C_{14}</td><td>450 / 450</td><td>12 Ø20</td></tr>
<tr><th colspan="4">Building B</th></tr>
<tr><th>Typical Plan</th><th colspan="3">Column Details</th></tr>
<tr><td rowspan="4">
</td><th>Column</th><th>Dimensions (mm)</th><th>Reinforcement (mm)</th></tr>
<tr><td>C_{1}, C_{2}, C_{3}, C_{4}, C_{5}, C_{6}, C_{19}, C_{20}, C_{21}, C_{22}, C_{23}, C_{24}</td><td>750 / 400</td><td>8 Ø16</td></tr>
<tr><td>C_{7}, C_{12}, C_{13}, C_{18}</td><td>400 / 750</td><td>8 Ø16</td></tr>
<tr><td>C_{8}, C_{9}, C_{10}, C_{11}, C_{14}, C_{15}, C_{16}, C_{17}</td><td>650 / 650</td><td>8 Ø20</td></tr>
</table>

Table 3. Service axial load and limiting shear strength for the first storey columns of Building A

Plan Direction	**Column**		**Axial Load**		**Shear Resistance**				
			N (kN)	**ν**	V^c_{flex} (kN)	V^c_{shear} (kN)	V^c_{anch} (kN)	V^c_{joint} (kN)	$V^c_{min,ele}$ (kN)
X	C_1, C_5, C_{16}, C_{20}	Top	-511	0.20	66.02	51.55	29.23	93.76	29.23
		Bottom	-527	0.21	66.93	52.05	56.53	-	
	C_2, C_4, C_{17}, C_{19}	Top	-1175	0.47	122.40	69.67	67.20	93.76	67.20
		Bottom	-1191	0.47	123.30	70.05	106.08	-	
	C_3, C_{18}	Top	-1044	0.41	114.93	66.50	59.73	93.76	59.73
		Bottom	-1060	0.40	115.84	66.89	98.62	-	
	C_6, C_{10}, C_{11}, C_{15}	Top	-549	0.22	52.78	33.78	19.14	60.03	19.14
		Bottom	-565	0.22	53.33	34.09	40.86	-	
	C_7, C_9, C_{12}, C_{14}	Top	-1398	0.37	153.54	95.46	79.95	93.76	79.95
		Bottom	-1422	0.38	154.90	96.07	130.87	-	
	C_8, C_{13}	Top	-1204	0.32	142.46	90.28	68.87	93.76	68.87
		Bottom	-1228	0.32	143.82	90.93	119.79	-	
Y	C_1, C_5, C_{16}, C_{20}	Top	-511	0.20	40.23	33.00	17.81	60.03	17.81
		Bottom	-527	0.21	40.79	33.33	32.48	-	
	C_2, C_4, C_{17}, C_{19}	Top	-1175	0.47	74.59	44.61	40.95	60.03	40.95
		Bottom	-1191	0.47	75.14	44.85	62.67	-	
	C_3, C_{18}	Top	-1044	0.41	70.03	42.58	36.40	60.03	36.40
		Bottom	-1060	0.40	70.59	42.83	58.12	-	
	C_6, C_{10}, C_{11}, C_{15}	Top	-549	0.22	86.61	52.76	31.41	93.76	31.41
		Bottom	-565	0.22	87.51	53.25	70.29	-	
	C_7, C_9, C_{12}, C_{14}	Top	-1398	0.37	153.54	95.46	79.95	125.02	79.95
		Bottom	-1422	0.38	154.90	96.07	130.87	-	
	C_8, C_{13}	Top	-1204	0.32	142.46	90.28	68.87	125.02	68.87
		Bottom	-1228	0.32	143.82	90.93	119.79	-	

Application of the proposed methodology indicated that the main problem of the first storey columns in all of the cases was the insufficient anchorage of the longitudinal reinforcement in the joint region at the column top, which limited the column strength to very low levels. Thus, the total base shear that Building A could sustain was 1039.28 kN in X direction and 891.02 kN in Y direction, while Building B could sustain 2321.61 kN and 1989.50 kN in X and Y direction, respectively. Given that the axial loads of the first storey columns for the G+0.3·Q load combination were 19396 kN for Building A and 12172 kN for Building B, Building A could sustain a peak ground acceleration (PGA) of only 5% of g, whereas Building B 16% of g. According to field recordings of the 1999 earthquake, the peak ground acceleration at the region of the buildings was 38% of g, which was extremely high for the level of detailing for the two buildings that collapsed.

Table 4. Service axial load and limiting shear strength for the first storey columns of Building B

Plan Direction	Column		Axial Load		Shear Resistance				
			N (kN)	ν	V^c_{flex} (kN)	V^c_{shear} (kN)	V^c_{anch} (kN)	V^c_{joint} (kN)	$V^c_{min,ele}$ (kN)
X	C_1, C_6, C_{19}	Top	-389	0.06	165.47	324.24	83.75	354.16	83.75
		Bottom	-406	0.07	165.47	324.24	165.47	-	
	C_2, C_5, C_{20}	Top	-662	0.11	224.25	363.61	142.52	354.16	142.52
		Bottom	-679	0.11	224.25	363.61	224.25	-	
	C_3, C_4, C_{21}, C_{22}	Top	-523	0.09	194.32	344.13	112.60	354.16	112.60
		Bottom	-540	0.09	194.32	344.13	194.32	-	
	C_7, C_{12}, C_{13}, C_{18}	Top	-518	0.09	135.90	176.70	59.59	182.24	59.59
		Bottom	-535	0.09	135.90	176.70	135.16	-	
	C_8, C_{11}, C_{14}, C_{17}	Top	-811	0.10	266.16	363.76	135.77	260.12	135.77
		Bottom	-839	0.10	266.16	363.76	227.49	-	
	C_9, C_{10}, C_{15}, C_{16}	Top	-600	0.07	233.14	341.20	102.74	260.12	102.74
		Bottom	-628	0.07	233.14	341.20	194.47	-	
Y	C_1, C_6, C_{19}	Top	-389	0.06	122.17	166.84	41.39	182.24	41.39
		Bottom	-406	0.07	122.17	166.84	121.43	-	
	C_2, C_5, C_{20}	Top	-662	0.11	151.22	187.09	74.91	182.24	74.91
		Bottom	-679	0.11	151.22	187.09	150.48	-	
	C_3, C_4, C_{21}, C_{22}	Top	-523	0.09	136.43	177.07	60.12	182.24	60.12
		Bottom	-540	0.09	136.43	177.07	135.69	-	
	C_7, C_{12}, C_{13}, C_{18}	Top	-518	0.09	193.24	343.41	111.52	354.16	111.52
		Bottom	-535	0.09	193.24	343.41	193.24	-	
	C_8, C_{11}, C_{14}, C_{17}	Top	-811	0.10	266.16	363.76	135.77	346.83	135.77
		Bottom	-839	0.10	266.16	363.76	227.49	-	
	C_9, C_{10}, C_{15}, C_{16}	Top	-600	0.07	233.14	341.20	102.74	346.83	102.74
		Bottom	-628	0.07	233.14	341.20	194.47	-	

Conclusions

A methodology for prioritizing the potential failure mechanisms in the load carrying system of concrete buildings classified as "non-conforming" according to modern standards was presented in this paper. Mechanisms considered refer to column flexure, shear, anchorage lap/splice development capacity and joint shear, as failure of vertical structural elements is directly related to building severe damage or collapse. The methodology can be applied to every column of the building, regardless of its location; if flexural strength may be supported by the shear, anchorage and joint resistance mechanisms, the calculated limiting strength can be even compared to the developing shear force derived from a seismic analysis. However, in most cases of existing structures, brittle failure modes are prioritized to occur prior to flexural yielding, so that no ductility may be realized, whereas the building collapses at displacements lower than the yield point. In this case, the methodology may also be applied for the determination of the maximum ground acceleration that the building can sustain, as illustrated it the former examples, as a tool of rapid evaluation of the buildings' seismic vulnerability.

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