Performance of RC buildings in the 2023 Turkey-Syria earthquakes

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ABSTRACT

The 2023 M_w 7.7-7.8 Turkey-Syria earthquakes were devastating events that resulted in widespread damage throughout several regions of south-eastern Turkey. In March 2023, the Architectural Institute of Japan deployed a team of 13 Japanese researchers, accompanied by 16 Turkish researchers to investigate the reinforced concrete building damage observed throughout the heavily damaged regions of Hatay, Kahramanmaraş, Gaziantep, Adiyaman and Malatya. The group undertook a generic widespread damage observations targeting high building volumes, as well as detailed investigation of 25 buildings. In the detailed assessment, building plan sketches, member size measurements, visual damage assessments and microtremor measurements were recorded. The Japanese damage assessment methodology was used to calculate the seismic residual capacity ratio of each building using this information. In this paper the results of the detailed assessment are reported. Additionally, commonly observed building construction characteristics and reinforcement detailing are reported and compared to standards and practices in New Zealand.

1 BACKGROUND

On 6th February 2023 a M_w 7.8 earthquake struck the southern provinces of Turkey, resulting in severe shaking to nearby populated cities. Several hours later, a second equally severe M_w 7.7 earthquake occurred approximately 100 km from the initial earthquake. The earthquakes resulted in widespread damage buildings and infrastructure in the surround provinces. In response to this disaster, a detailed survey of reinforced concrete (RC) buildings was undertaken by a team of researchers from Japan and Turkey approximately two months following the earthquakes. The team members, listed in Table 1, were split into multiple teams such that each team had a combination of Japanese and Turkish researchers. The survey was conducted in five provinces shown in Figure 1: Gaziantep, Hatay, Kahramanmaraş, Adiyaman, and Malatya. These areas were chosen as they were most densely built and had widespread building damage and collapse. At the time of the survey, while demolition was still underway, the enormous scale of the damage meant that a large number of damaged and collapsed buildings were still present. Most buildings in the heavily damaged regions had been abandoned and emptied by the residents, making investigations easier to undertake. In this paper, results of the detailed survey are presented and comparisons are made against buildings in Japan. Some comparisons are also drawn to common construction practices in New Zealand.

Table 1: List team members participating in the earthquake damage survey.

Japan team	Turkish team
Tatsuya Asai (University of Tokyo) Naoki Onishi (University of Tokyo) Koichi Kusunoki (University of Tokyo) Kuniyoshi Sugimoto (Yokohama National University) Seitaro Tajiri (University of Tokyo) Akihiro Nakamura (BRI) Masaki Maeda (Tohoku University) Maruya Satoshi (Japan Architectural Center) Yoon Rokhyun (Osaka University) Hong Liu (Tokyo University of Science) Jonathan Monical (Tohoku University) Mohammad Mahdi Raouffard (Taisei Corperation) Alex Shegay (Tokyo Institute of Technology)	Alper Ilki (Istanbul Technical University) Ufuk Yazgan (Istanbul Technical University) Tolga Cinar (Istanbul Technical University) Cem Demir (Istanbul Technical University) Cem Yenidogan (Yildiz Technical University) Caglar Akkaya (Istanbul Technical University) Goktug Unal (Istanbul Technical University) Fehmi Dogan (Izmir Institute of Technology) Ugur Demir (Izmir Institute of Technology) Huseyin Aydogdu (Istanbul Metropolitan Municipality) Kurtulus Atasever (Mimar Sinan Fine Arts University) Omer Halici (MEF University) Hilmi Lus (Bogazici University) A. Alperen Koc (Bogazici University) Kutay Orackal (Bogazici University) Cem Tura (Bogazici University) M. Fethi Güllü (Harran University)

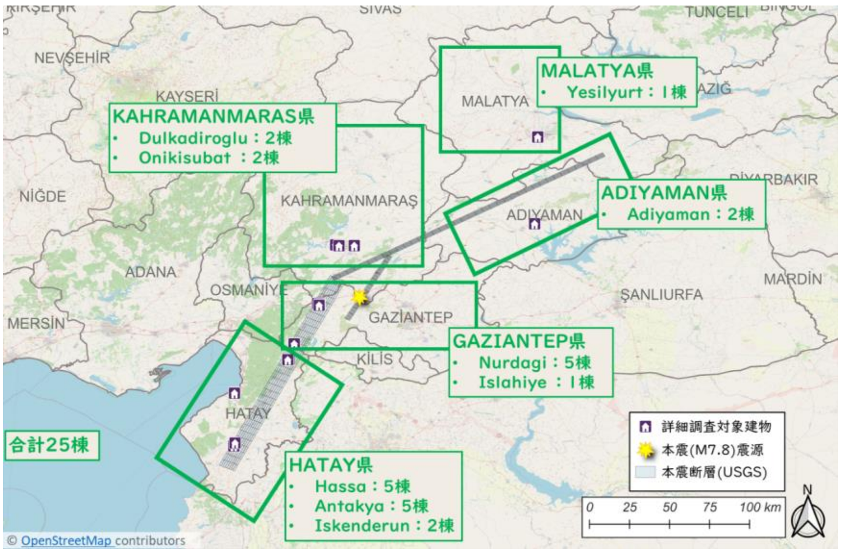

Figure 1: Spatial distribution of surveyed buildings.

2 TARGET BUILDINGS AND SURVEY OBJECTIVES

The buildings surveyed were predominantly low- to mid-rise reinforced concrete buildings. The criteria for selecting the target buildings for detailed investigation included the following:

1. Building is suitable for comparative analysis following the investigation. For example, buildings in the same vicinity and similar designs characteristics but exhibiting different types or severity of damage.
2. The building is easily accessible so that sufficient detail can be gathered in the survey (e.g., buildings where entry and survey permission is easily obtained, buildings where the arrangement and sectional dimensions of columns and walls can be easily measured). Often this narrowed it down to buildings that were either under construction and near completion when the earthquake struck, or those that have been stripped of internal partitions.
3. Building is safe to enter – i.e., not collapsed or at imminent risk of collapse.

Based on these criteria, data from a total of 25 buildings were collected over several days. The full list of buildings is provided in Table 2, and images for each one is shown in Figure 2. As can be seen from the table, most buildings were new or recently completed (post-2000).

For each buildings the following data were collected:

- Building use type
- Main failure mode of building and any other key damage characteristics
- Hand sketches of the plan, including position and dimensions of all vertical elements.
- Photos of all members and critical damage locations
- The assessed damage level of columns, beams, and walls at the most damaged floor (usually the ground floor). In some cases, damage states of upper floors were also noted.
- For some buildings, measuring the dimensions of masonry walls on the most damaged floors and assessing their damage level.

Figure 2: Photos of surveyed buildings.

Table 2: List of RC buildings subjected to detailed investigation.

ID*	Number of floors	Construction year	Major damage	*R* (%) X	*R* (%) Y	Judgement‡
N1	7	UC†	Crushing at column bases	-	-	-
N2	6	UC	Crushing at column bases	56	56	Severe
N3	7	UC	Crushing at column bases	74	68	Moderate
N4	7	Unknown	Wall shear failure Column base crushing	80	50	Moderate
N5	7	UC	Beam flexural damage	75	81	Moderate
S1	4	~2008	Shear failure of short column	39	62	Severe
H1	6	~2010	Shear failure of columns	94	99	Slight/No
H2	6	Post-2005	Crushing at column bases	-	78	Moderate
H3	9	UC	Beam flexural damage	75	73	Moderate
H4	9	UC	Beam flexural damage	87	88	Minor
H5	8	UC	Wall shear failure	75	78	Moderate
I1	7	Unknown	Column base crushing	51	59	Severe
I2	5	UC	Essentially undamaged	100	100	Slight/No
A1	9	UC	Crushing of end columns Shear failure of perimeter beams	84	84	Minor
A2	9	UC	Beam flexural damage	17	5	Severe
A3	7	Unknown	Column flexural damage	53	39	Severe
A4	7	UC	Coupling beam failure Beam flexural failure	66	53	Severe
A5	C:13 D:8	UC	C: Beam flexural failure D: Beam flexural failure	C:47 D:71	C:31 D:60	Severe
K1	10	1997	Column shear damage	52	54	Severe
K4	11	Unknown	Coupling beam failure Wall crushing failure	39	76	Severe
K2	13	2011	Shear failure of coupling beams	94	100	Slight/No
K3	12	2020	Wall shear failure	61	68	Moderate
D1	5	Post-2020	Column base crushing	84	96	Minor
D2	5	Post-2020	Slight cracking	99	99	Slight/No
M1	13	Unknown	Slight cracking	97	99	Slight/No

* Building name assigned by the survey team.
† UC: Under construction.
‡ Based on the average of the two directions, and where $R \geq 95\%$ for Slight, $95 \geq R > 80$ for Minor, $80 \geq R > 60$ for Moderate and $R < 60$ for Severe.

3 DAMAGE LEVEL CLASSIFICATION

Based on the damage level classification criteria described in the Guidelines produced by the Japan Building Disaster Prevention Association (JBDPA Guidelines, 2015), the damage level of columns and RC walls on the most damaged floors was assessed. Using this data, the seismic residual capacity ratio, R, was subsequently calculated. Details of the damage classification criteria and calculation method of R are provided next.

3.1 Damage Level Assessment

The damage level of columns, beams, and RC walls on the surveyed floors was determined in accordance with the guidelines for damage level classification shown in Table 3, in both the column and beam directions of the building.

Table 3: Definition of damage levels as per the JBDPA Guidelines (JBDPA, 2015).

Damage Level	Observed damage in structural members	
	Flexure governed response	Shear governed response
I	Sparse, fine cracks can be observed (<0.2 mm).	No reinforcement yielding expected.
II	Clearly visible cracks (0.2 - 1 mm) exist.	Visible diagonal cracks (shear cracks) are apparent to the naked eye, with crack widths ranging from approximately 0.2 to 1 mm.
III	Wide cracks (1 - 2 mm) are present. Plastic hinging mechanisms begin to form. Some spalling of cover concrete is observed but concrete core is in-tact.	Wide cracks (1 - 2 mm) are present but there is very little spalling of the cover concrete, and the core concrete remains sound, with no reduction in strength.
IV	Many wide cracks are observed. Compression damage resulting in concrete spalling and exposed reinforcement. Lateral strength degradation may occur, but vertical load is still fully carried by walls and columns.	Many wide cracks (1 - 2 mm) are present. There is significant spalling of the cover concrete, some compression failure, and exposed reinforcement bars. Lateral strength degradation may occur but no damage (buckling or fracture) to the longitudinal and shear reinforcement.
V	Buckling (and in some cases fracture) of reinforcement, crushing of concrete and vertical deformation of columns and/or shear walls observed. Settlement and inclination of structure are characteristic.	

3.2 Residual capacity ratio calculation

In the JBDPA Guidelines, two methodologies are outlined for estimating the seismic residual capacity ratio, R: detailed method and simplified method. Since the information collected from the building surveys was limited (e.g., cross section details or material properties were sparse), the simplified methodology is adopted here. Equation 1 below shows the method for calculating the simplified seismic residual capacity ratio. It is noted that the calculation method procedure changes depending on whether a beam-sway (total collapse) or column-sway (storey-collapse) mechanism is expected to form. In this paper, a storey-collapse mechanism was conservatively assumed for all buildings.

$$R = \frac{\sum_{j=0}^{5} A_j}{A_{org}} \times 100 \quad (\%) \qquad (1)$$

$$A_j = k_c \sum {}_S\eta_j S_j + k_c \sum {}_{SM}\eta_j SM_j + k_c \sum {}_M\eta_j M_j + k_c \sum {}_{SB}\eta_j SB_j + k_c \sum {}_{MB}\eta_j MB_j + k_w \sum {}_W\eta_j W_j + k_{cw} \sum {}_{CW}\eta_j CW_j + k_{cwc} \sum {}_{CWC}\eta_j CWC_j \tag{2}$$

$$A_{org} = k_c S_{sum} + k_c SM_{sum} + k_c M_{sum} + k_c SB_{sum} + k_c MB_{sum} + k_w W_{sum} + k_{cw} CW_{sum} + k_{cwc} CWC_{sum} \tag{3}$$

In the above equation, A_j: normalized residual capacity of members (column or beam) on a selected floors with damage class j in the selected building direction, A_{org}: normalized residual capacity of members (column or beam) on a selected floors in the selected building direction, k_c, k_w, k_{cw}, k_{cwc}: strength index of column (=1), wall (=1), wall with single end column (=2) and wall with two end columns (=6), respectively. S_j, SM_j, M_j, SB_j, MB_j, W_j, CW_j, CWC_j: number shear columns, flexure-shear columns, flexural columns, columns with shear governed beams, column with flexure governed beams, wall, wall with single end column and wall with two end columns, with damage class j; ${}_S\eta_j$ ${}_{SM}\eta_j$ ${}_M\eta_j$ ${}_{SB}\eta_j$ ${}_{MB}\eta_j$ ${}_W\eta_j$ ${}_{CW}\eta_j$ ${}_{CWC}\eta_j$: strength reduction factor for the respective elements, and S_{sum}, SM_{sum}, M_{sum}, SB_{sum}, MB_{sum}, W_{sum}, CW_{sum}, CWC_{sum}: sum of respective elements (irrespective of damage class).

3.3 Adjustments to suit Turkish building practice

The strength indices defined in the JBPDA Guidelines (i.e., the 1:1:2:6 for column, walls, walls with single column and walls with two end columns, respectively), have been set based on the typical section shapes used in Japanese construction practice. After surveying several Turkish buildings, it became clear that common dimensions of vertical members in Turkey are distinctly different from those found in Japan. Unlike in Japan, where columns are often square in cross-section, in Turkey, the columns are an elongated cross-section, resembling a short wall. Therefore, the columns in Turkish buildings would have asymmetric flexural strength characteristics in the two orthogonal directions. Since the residual capacity ratio in Equation 1 assumes square columns, the differences in flexural strengths in the two orthogonal directions for Turkish members needed to be accounted for through corrections to the strength indices.

Firstly, it was necessary to address the classification of vertical members into columns and walls. In RC buildings in Turkey, the distinction between walls and columns was challenging due to the elongated rectangular cross-sections. Furthermore, unlike in Japan, columns at the ends of walls (i.e., barbell walls) are not commonly used. To enable Equation 1 to be applied, the following simple criteria were established to distinguish between walls and columns:

- Vertical members with a long side dimension of <2 meters and with cross-sectional shapes close to a square were classified as columns.
- Vertical members with a long side dimension of >2 meters or comprising a core of a building were classified as walls.

Next, it was necessary to define new strength indices to account for the fact that slender rectangular columns have a lateral strength capacity that is larger about the strong axis than the weak axis and both are different than if the column had a square cross-section. In the primary diagnosis of seismic assessment standards, the strength of a column is roughly taken as τ_c = 1.0 MPa, and is reduced to τ_c = 0.7 MPa for columns where the ratio of the clear height to the column width ratio h_0/D is 6 or more (i.e., slender columns). Following this logic, for slender columns (where h_0/D is 6 or more), it was decided to reduce the strength index k_c from 1 to 0.7. Conversely, for columns subjected to horizontal forces about the strong axis, it is considered that they can bear larger shear stresses than square cross-section columns. Following the seismic assessment standards mentioned above, for extremely short columns (where h_0/D is 2 or less), τ_c is increased to 1.5 MPa; thus, it was decided to increase k_c from 1 to 1.5 for columns where h_0/D is 2 or less.

Finally, the differences of typical wall cross-sections in Japan and Turkey were addressed. Unlike Japan, RC walls in Turkey rarely have columns at the end regions. However, the detailing in the end regions of the walls is somewhat representative of what would be found in Japanese barbell walls – characterized by closely spaced transverse confining reinforcement. For this reason, the strength index chosen for rectangular walls in Turkey is taken as mid option for the three walls type classifications possible in Equation 1, i.e., wall with single end column (k_{cw} = 2). For out-of-plane direction, the strength index for a slender column loaded about the weak axis adopted (k_{cw} = 0.7). In summary, the adjusted strength index values are shown in Table 4.

Table 4: Summary of adjustments made to the strength index.

Classification		Before adjustment	Adjusted
Column	$h_0/D \geqq 6$ (Slender column)	$k_c = 1$	$k_c = 0.7$
	$2 < h_0/D < 6$	$k_c = 1$	$k_c = 1$
	$h_0/D \leqq 2$ (Stocky column)	$k_c = 1$	$k_c = 1.5$
Wall	In-plane	$k_w = 1$, $k_{cw} = 2$, $k_{wcw} = 6$	$k_{cw} = 2$
	Out-of-plane	-	$k_{cw} = 0.7$

4 RESULTS

4.1 Example of Survey Results

An example of the calculation of R is presented for the H1 building shown in Figure 1. H1 an RC 6-storey residential apartment building built in 2010 and located in Hassa city of the Hatay province. A detailed survey of the ground floor was conducted, collecting information described in section 2. Figure 3 shows the layout and dimensions of the columns and walls at the ground floor as measured on site.

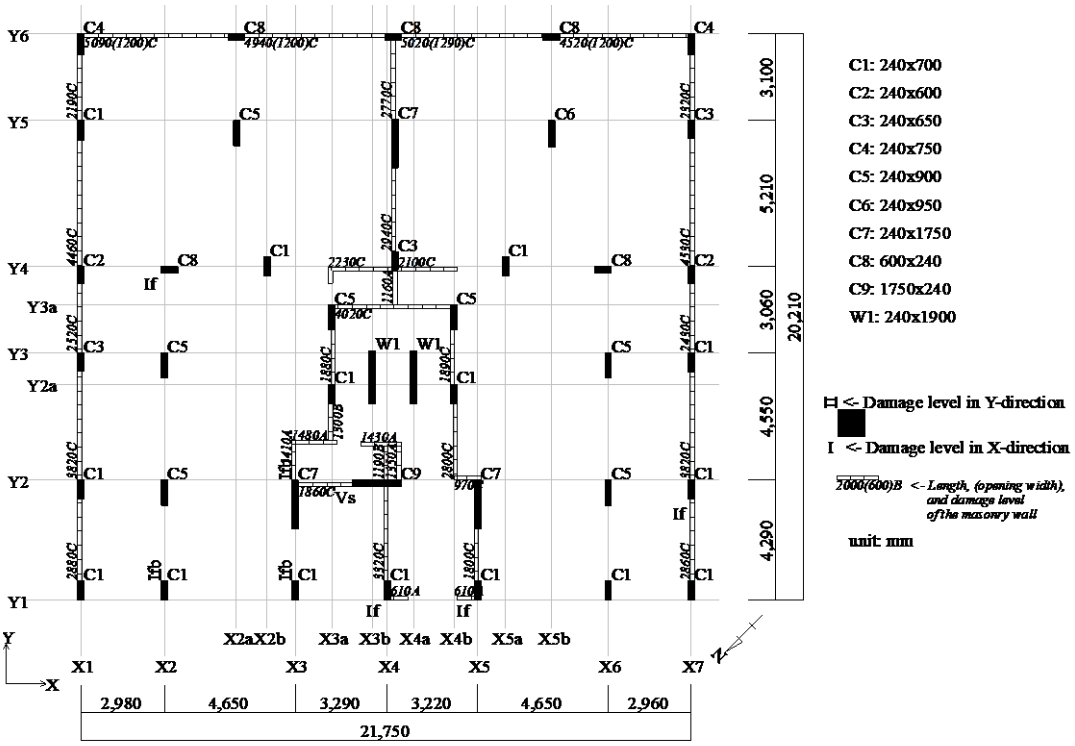

Figure 3: Plan drawing of H1, including damage state of vertical members.

The building has a setback in the elevation direction, with the sections between X1–X2 and X6–X7 being one floor above ground, and the section between X2–X6 being six floors above ground. The elements classified as columns and walls according to the criteria of section 3.3 are labelled as 'C' and 'W', respectively. The damage levels (in the X and Y directions) of each vertical member columns and walls, judged as level I or higher according to Table 3, are also indicated in the figure (where roman numerals refer to damage level and the 'f' and 's' refer to shear or flexure dominant response). The main structural damage observed in this building was the shear failure of column C9 at X4Y2.

Based on these damage levels, an example calculation of the R for the X direction is shown in Table 5. It was determined that the residual capacity ratio in the X-direction was 29.345/31 ×100 = 94%, which corresponds to 'minor' damage (JBDPA, 2015). Similarly, in the Y direction, R was determined to be 99%, which corresponds to negligible damage.

Table 5: Residual capacity calculation for H1 in the X-direction.

	Column			Long column			Short column			Beam		Wall			
	Shear	Flexure and Shear	Flexure	Shear	Flexure and Shear	Flexure	Shear	Flexure and Shear	Flexure	Shear	Flexure	In-plane	Out-of-plane	Total	
No. total members	( )	( )	( )	( )	( )	( )	( )	( )	( )	( )	( )	( )	( )	(41)	
No. investigated members	( )①	( )②	(5)③	( )④	( )⑤	(33)⑥	(1)⑦	( )⑧	( )⑨	( )⑩	( )⑪	( )⑫	(2)⑬	(41)	
	①×1	②×1	③×1	④×0.7	⑤×0.7	⑥×0.7	⑦×1.5	⑧×1.5	⑨×1.5	⑩×1	⑪×1	⑫×2	⑬×0.7	(31)	$=A_{org}$
Damage degree 0	( )	( )	(4)	( ) ×0.7	( ) ×0.7	(30) ×0.7	( ) ×1.5	( ) ×1.5	( ) ×1.5	( )	( )	( ) ×2	(2) ×0.7	(26.4)	$=A_0$
Damage degree I	( ) ×0.95	( ) ×0.95	(1) ×0.95	( ) ×0.665	( ) ×0.665	(3) ×0.665	( ) ×1.425	( ) ×1.425	( ) ×1.425	( ) ×0.95	( ) ×0.95	( ) ×1.9	( ) ×0.665	(2.945)	$=A_1$
Damage degree II	( ) ×0.6	( ) ×0.7	( ) ×0.75	( ) ×0.42	( ) ×0.49	( ) ×0.525	( ) ×0.9	( ) ×1.05	( ) ×1.125	( ) ×0.7	( ) ×0.75	( ) ×1.2	( ) ×0.42	(0)	$=A_2$
Damage degree III	( ) ×0.3	( ) ×0.4	( ) ×0.5	( ) ×0.21	( ) ×0.28	( ) ×0.35	( ) ×0.45	( ) ×0.6	( ) ×0.75	( ) ×0.4	( ) ×0.5	( ) ×0.6	( ) ×0.21	(0)	$=A_3$
Damage degree IV	( ) ×0	( ) ×0.1	( ) ×0.2	( ) ×0	( ) ×0.07	( ) ×0.14	( ) ×0	( ) ×0.15	( ) ×0.3	( ) ×0.1	( ) ×0.2	( ) ×0	( ) ×0	(0)	$=A_4$
Damage degree V	( ) ×0	( ) ×0	( ) ×0	( ) ×0	( ) ×0	( ) ×0	(1) ×0	( ) ×0	( ) ×0	( ) ×0	( ) ×0	( ) ×0	( ) ×0	(0)	$=A_5$

$$\Sigma A_j = A_0 + A_1 + A_2 + A_3 + A_4 + A_5 = 29.345$$

4.2 Overview of Survey Results

The residual capacity ratio was calculated for every building in Table 2, and the results are summarized in the same table. There are almost half of the buildings judged as severely damaged, and a similar number of buildings judged as moderately damaged, slightly damaged, and minorly damaged, indicating that a wide range of damage levels was covered in the survey. It is evident that even buildings built recently, or still in construction did not necessarily have superior performance. Additionally, while the range of building heights covered in this study is limited and generally focused on mid- to high-rise (4-13 storeys), from this dataset it appears that no building height experienced particularly damage characteristic.

Finally, one of the most common damage patterns observed in the surveyed buildings was the 'apparent' buckling of longitudinal reinforcement in the vertical members at the bottom storey. The term 'apparent' is used because standard practice in Turkey is to bend starter longitudinal reinforcement rising from the foundation into the member core, as shown in Figure 4. These observations made it difficult to distinguish genuine buckling from intentional formation; thus, damage judgements according to Table 3 may have been overly penalizing in some situations. It is noted that in the most recent version of the Turkish code (TBSC, 2018), splicing in column hinges is required to be installed in the central region of the column (i.e., in region of low moment demands and away from the plastic hinge). In New Zealand, longitudinal reinforcement splices are also not permitted in plastic hinges of columns, so the problem of 'apparent' buckling is unlikely to arise (Standards New Zealand, 2017). This widespread practice of bending rebar in Turkey warrants investigation to determine the effect this has on flexural strength and deformation capacity of plastic hinges.

Figure 4: Intentionally bent longitudinal starter bars appearing as 'buckling'.

4.3 Characteristics of Buildings and Damage

Several typical features of Turkish buildings were noted to try generalize performance of buildings. These are summarized below.

- The majority of the buildings use flat section columns (approximately 250–300 mm by 600–2000 mm). Buildings in New Zealand typically use similar column dimensions in orthogonal directions. Many of the buildings have relatively short span widths (approximately 4–5 m)
- Many cases of concrete crushing and shear failure at the base of ground-floor columns were observed, despite closely spaced transverse reinforcement
- Starter bars from the foundation are often bent off the vertical to accommodate splicing reinforcement from the vertical member. This discontinuity at the critical section was often where crushing of concrete was observed. In New Zealand, splices in plastic hinge regions of columns are generally not permitted
- Many buildings had more damage concentrated on beams than columns, with damage concentration also observed on short-span beams
- Shear and bending failures were frequently observed in RC core walls, the reinforcement detailing of which was often aligned with New Zealand practice
- Even if structural damage was minor, many buildings had significant damage (crushing, toppling) to the light-weight unreinforced masonry blocks walls that are often used as partitions. This type of damage can prevent the space from being inhabitable; thus, emphasizing the importance of consideration of 'whole-of-building' response when targeting performance targets
- Insufficient cover and honeycombing were observed even in newer buildings. Schmidt hammer testing on several buildings indicated strengths around 20 MPa. This is generally deemed satisfactory for Turkish buildings constructed prior to 2000, as strengths as low as 10 MPa were common during those periods. These strengths are generally lower than typical strengths used in NZ (≥30 MPa)

- Buckling of longitudinal reinforcement and spalling of concrete were seen in the beam-column joints of flat columns, in regions not restrained by beams
- Cases were observed where column shear spans shortened due to restraint by unreinforced masonry walls, resulting in a captive column effect
- Signs of pounding of adjacent buildings was observed in several cases
- Diaphragms are predominantly made up of either flat slabs, or a system of masonry blocks stacked between small spandrel beams and integrated with a topping slab. Buildings utilizing the latter generally had more severe overall damage than those with the latter. Unlike in NZ, precast floor systems were not prevalent.

5 CONCLUSION

The 2023 Turkey-Syria earthquakes highlighted various vulnerabilities in the region's building stock, with a collaborative survey between Japanese and Turkish researchers revealing a range of damage levels across various RC structures, rendering buildings uninhabitable. Despite being of recent construction, many buildings did not demonstrate superior seismic performance, with common issues like concrete crushing at column bases, widespread masonry partition damage and inadequate splice detailing. While there are some notable construction practice differences compared to New Zealand practice, similarities in other reinforcement detailing aspects in walls and columns suggest potentially similar performance can be expected in New Zealand RC buildings. These observations underscore the need for improving building design philosophies such that buildings can be immediately occupiable on top of satisfying the life-safety objectives.

6 ACKNOWLEDGEMENTS

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