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PERFORMANCE OF REINFORCED CONCRETE BUILDINGS IN THE 2016 KUMAMOTO EARTHQUAKES AND SEISMIC DESIGN IN JAPAN

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ABSTRACT

This report outlines the observations of an NZSEE team of practitioners and researchers who travelled to the Kumamoto Prefecture of Japan on a reconnaissance visit following the April 2016 earthquakes. The observations presented in this report are focussed on the performance of reinforced concrete (RC) buildings throughout Kumamoto Prefecture. It was found overall that modern RC buildings performed well, with patterns of damage which highlighted a philosophy of designing stiffer buildings with less of an emphasis on ductile behaviour. To explore this important difference in design practice, the Japanese Building Standard Law (BSL) is summarised and compared with standard New Zealand seismic design practices and evaluation methods.

INTRODUCTION

Between 14 April and 16 April, 2016, a series of earthquakes struck Kumamoto Prefecture on Kyushu Island, Japan (Figure 1). An NZSEE reconnaissance team made up of structural engineering researchers and practitioners (the authors), in collaboration with a Japanese team funded by Japan Science and Technology Agency, visited the Kumamoto Prefecture between 29 June and 3 July 2016. This report is organized to provide insights on the seismic demands from the earthquake and observed building performance, followed by a summary of seismic design actions, assessment procedures for existing buildings, and post-earthquake damage assessment procedures used in Japan. Detailed reports on building performance are provided in Appendices A1-A16 and referenced throughout the body of the paper.

KUMAMOTO EARTHQUAKE SEQUENCE

The largest foreshock of the Kumamoto sequence occurred at 21:26 JST on 14 April, with a magnitude of M_j 6.5, in the Northern regions of the Hinagu Fault. This event was followed by further seismic activity, including the magnitude M_j 7.3 main-shock at 01:25 JST on 16 April due to right lateral strike-slip movement of the Futagawa Fault. The main-shock was approximately 15 km South-East of Kumamoto City at a depth of 12 km [1]. Magnitudes are quoted in M_j, which is the local magnitude scale calculated by the Japanese Meteorological Agency (JMA). In Mashiki near the Hinagu and Futagawa faults, both major events resulted in the maximum possible intensity of 7 on the Japan Meteorological Agency Intensity (JMAI) scale. Similar to the Canterbury Earthquake Sequence, the majority of the significant damage occurred in the second strong seismic event; but unlike Christchurch these events occurred so close together there was insufficient time to fully assess damage from the 14 April event before the 16 April event occurred just 28 hours later.

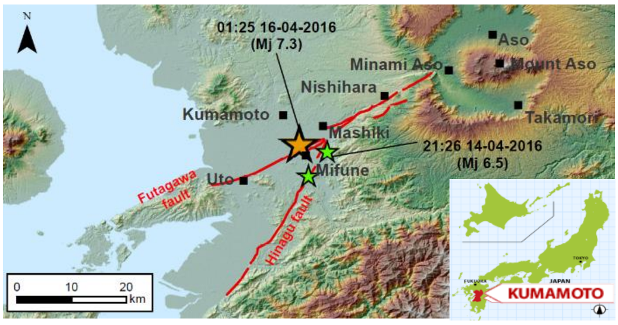

Figure 1: Epicentre and fault map of Kumamoto Prefecture. Adapted from Chiaro et al. (2017) [2]. Inset map of Japan obtained from https://www.kumagaku.ac.jp/english/ (accessed June 2017).

The 2016 April Kumamoto earthquakes resulted in 50 fatalities and almost 3000 injuries, with most deaths occurring due to collapse of residential houses concentrated in Mashiki Town to the east of Kumamoto [3]. Death toll and injuries would likely have been considerably higher if the 14 April event had not occurred since many people had already evacuated their houses at the time of the 16 April main-shock. Multistory reinforced concrete buildings generally performed very well, most enabling immediate occupancy after the earthquakes. Damaged multistory buildings were generally constructed prior to 1981, when the Japanese Building code was updated following the 1978 Miyagi Earthquake.

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Figure 2 provides a comparison of response spectra for the 14 and 16 April 2016 Kumamoto and 22 February 2011 Christchurch events. Above 0.7 sec, the three events resulted in very similar levels of shaking, but at short periods the maximum response for the Kumamoto event was approximately twice as large as the maximum response from the Christchurch earthquake. A comparison of the Christchurch and Kumamoto design spectra for soft ground is also shown in Figure 2. Kumamoto is a coastal region with a variety of ground conditions ranging from 'Hard' (defined as type 1 in Japanese building code) to 'Soft' (defined as type 3). The peak of the response spectrum for all three ground conditions is 0.9g. The spectrum for soft ground has been presented here to be consistent with the site class D spectrum shown for Christchurch. Japan Seismic Hazard Information Station (J-SHIS) publishes JMAI values for different return periods throughout Japan. Table 1 provides a comparison of JMAI values recorded in the 16 April event and probabilistic JMAI values for 475, 1000, and 2500 year return periods published on the J-SHIS website [4]. The comparison indicates that several stations close to the fault recorded ground motions at or above 2500 year motions. Kumamoto City experienced ground shaking representative of 500 to 1000 year motions.

MODERN BUILDING PERFORMANCE

Reinforced concrete construction has been widely used throughout Kumamoto City in government and school facilities as well as public and private apartment buildings. The most common structural system observed in residential reinforced concrete structures was moment frames with open first story, desirable by Japanese developers for maximizing the area usable for parking. Other common structural systems in residential buildings included combinations of moment frames in the long direction and shear walls in the transverse direction. Government and school facilities were commonly moment frames combined with shear walls. Base isolation was used in 24 buildings in the Kumamoto prefecture.

Based on observations of structures throughout the Kumamoto Prefecture it was evident that multi-story buildings were generally much stiffer and stronger than their New Zealand counterparts, with very few examples of modern buildings exhibiting damage consistent with high, or even moderate, ductility demands during the earthquake. Examples of such buildings are shown in Figure 3. Buildings with open first stories for parking were very common, most with negligible damage despite the appearance of a soft story. Examples of good performing buildings are detailed in Appendix A - Building Damage Reports A2, A5 & A6.

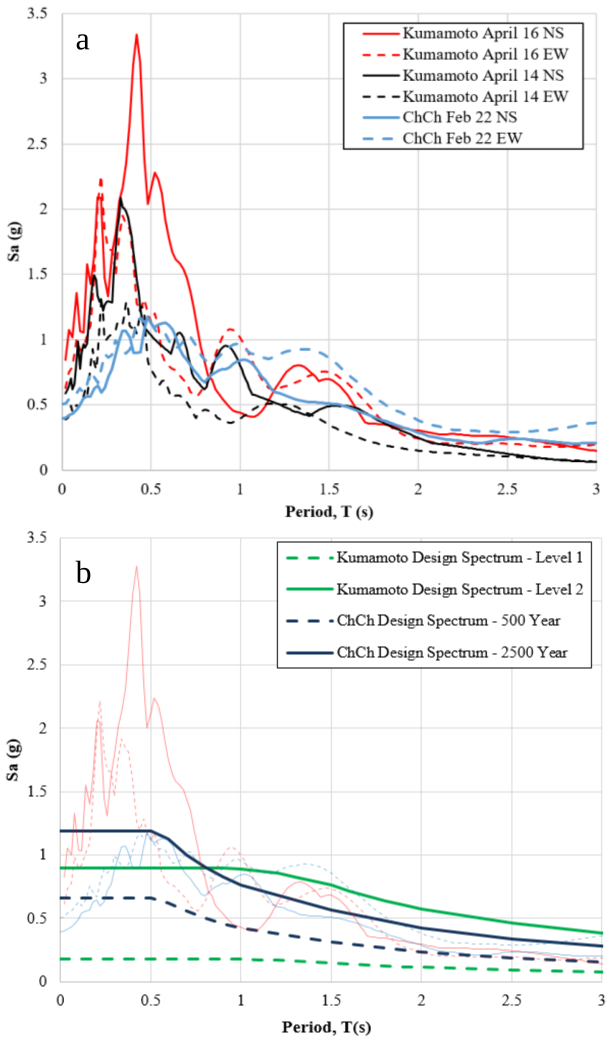


Figure 2: (a) Kumamoto vs. Christchurch response spectra. (b) Kumamoto vs. Christchurch design spectra (based on Z=0.22 – pre 2011 hazard factor). Response of April 16 Kumamoto and Feb. 22 Christchurch event included. Japanese design Level 1 represents the 'Damage Limit State' (similar to NZ SLS) and Level 2 represents the 'Safety Limit State' (similar to NZ ULS). Kumamoto response spectra obtained from data recorded at K-net station KMM006. Christchurch response spectra represent average response of data from CanNet stations CCCC and CHHC as well as NSMN stations CBGS and REHS.

Figure 3: Example of multistory buildings with no damage.

Table 1: Summary of recorded and probabilistic JMAI values for stations near Kumamoto. JMAI exceedance probabilities were obtained from www.j-shis.bosai.go.jp [4].

Location	Station ID	Station Distance from Epicentre (km)	JMAI Probability of Exceedance in Station Area			Kumamoto April 16th Event JMAI
			2% in 50 Yrs (1/2500)	5% in 50 Yrs (1/1000)	10% in 50 Yrs (1/500)	
Toyono	KMMH14	13.2	6 -/6 +	6 -	5 +/6 -	6 +
Mashiki	KMMH16	7.3	6 +/7	6 +	6 -	7
Yabe	KMM009	22.3	6 -	5 +/6 -	5 +	6 -
Uto	KMM008	12.1	7	6 +	6 +	6 +
Tomochi (Misato)	KMM011	18	6 -/6 +	5 +/6 -	5 +/6 -	6 -
Kumamoto	KMM006	4.7	7	6 +/7	6 +	6 +
Yatsushiro	KMM012	31.1	7	6 +/7	6 +	6 -
Izumi	KMMH09	32.2	6 -	5 +	5 +	4/5 -
Takamori	KMM007	34.7	6 -	6 -	5 +	5 +
Tanoura	KMM013	49.2	6 +/7	6 -/6 +	6 -	5 +
Kikuchi	KMMH03	28	6 -/6 +	6 -	6 -	6 -/6 +

These anecdotal observations are supported by the limited red and yellow placards applied to modern buildings in Kumamoto Prefecture. A summary of the buildings assessed in the Mashiki Town area showed that, of the assessed RC structures, 71% were constructed post 1981 of which 89% were assessed as undamaged with the remaining 11% being deemed as only moderately damaged. A common observation in modern multistory RC residential buildings was shear failure in "non-structural walls" (See appendix A2 and A8). These non-structural wall components were made of reinforced concrete and were commonly tied into the structural system with what Japanese designers referred to as "half connections". This connection is shown in Figure 4, where only half of the thickness of the wall has reinforcement extending into the lateral force resisting system. These walls are referred to as "non-structural" as they are not considered to contribute to the lateral strength of the building during design, but are accounted for when determining the stiffness of the structure. This practice is a contributing factor to the lower ductility behavior of the observed RC structures.

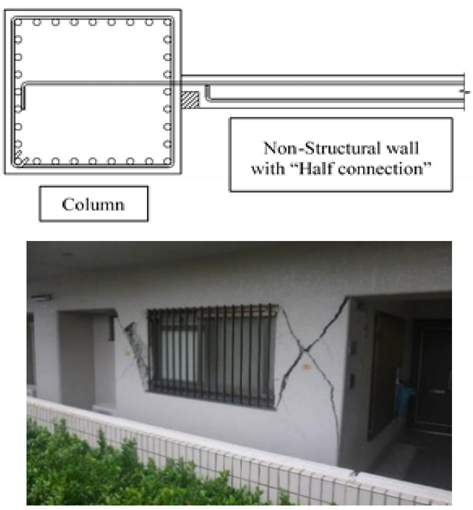


Figure 4: Typical non-structural wall connection and example of damage to non-structural wall (see Appendix A2).

Performance of Base Isolated Buildings

Three base isolated multistory RC residential buildings were inspected in close proximity to Kumamoto city centre. All three structures had some level of visible damage or residual displacement, but all had been cleared for occupation and had generally performed well. Details of the inspection of these buildings can be found in appendix A5 and A6. Figure 5 provides some examples of the external damage observed.

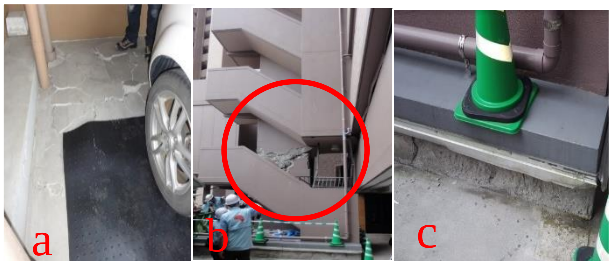


Figure 5: External damage to base isolated buildings (a) Base Isolated Building No.1 (Appendix A5) (b) & (c) Base Isolated Building No.2 (Appendix A6).

The reconnaissance team was given access to the basement of Base Isolated Building No. 1 (Appendix A5) to inspect the state of the isolation system of the structure. Observations of the damage (Figure 6) indicate that the building performed well, however due to some unconventional design choices, the damping system of the structure sustained heavy damage. The structure was damped using a combination of steel and lead dampers (refer to Appendix A5 for plan of isolation system and location of isolators). All base isolators were connected through a grid of concrete beams and no damage was observed to the isolators. The steel dampers were also connected into the concrete beam grid, however as shown in Figure 6, they were connected with an eccentricity which contributing to the damage sustained by the supporting concrete blocks. The lead dampers were completely excluded from the concrete beam grid and were simply connected to the concrete slab above by protruding concrete blocks. Most of the protruding concrete blocks for the lead dampers displayed similar damage to that shown in Figure 6 (c), where the concrete block-slab connection was completely lost.

Access was not obtained to the isolation basement of Base Isolated Building No.2 and No.3, however, the damage observed externally was minimal with the exception of shear failure of a stairwell wall which was concentrated between the first floor plane of isolation of the main structure and the base isolation of the stairwell at the ground level (Figure 5b). All base isolated buildings continued to be occupied after the earthquakes.

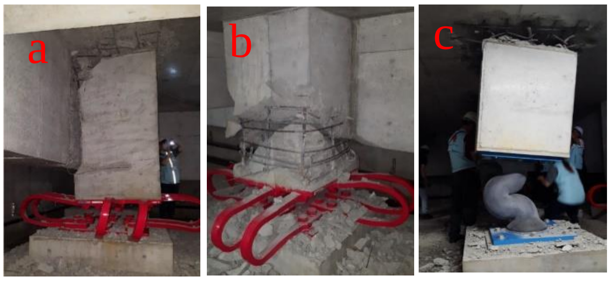


Figure 6: Damage sustained to Steel (a) & (b) and lead (c) dampers below Base Isolated Building No. 1.

Nonstructural Element Performance

Authors observed limited damage to nonstructural elements, in part due to clean up in the more than two months since the earthquakes but also due to design features used to minimize damage. There were many examples of mechanical plants being housed externally which avoided damage through the use of flexible connections and appropriate fixings. In cases of public school facilities, a lack of suspended ceilings and sprinkler systems was noted with the result being very limited amounts of damage due to non-structural system failure. Damage to suspended ceilings in gymnasium facilities was observed, and in one notable case resulted in emergency evacuation facilities not being suitable for use following the earthquake. It was noted by Japanese researchers, that several facilities in Japan have opted to remove suspended ceilings since the extensive damage to such systems in the 2011 Tohoku Earthquake. Further details of non-structural element performance can be found in appendix A16.

PRE-1981 CONCRETE BUILDING PERFORMANCE

Earthquake design in the Building Standard Law of Japan (BSL; equivalent to the New Zealand Building Statutes – including NZS1170) was significantly revised in 1981 following damage in the 1978 Miyagi Earthquake. As such, many pre-1981 buildings do not comply with the current seismic standard and have performed poorly in past earthquakes. Figure 7 provides three examples of pre-1980s concrete buildings with severe damage to columns and beam-column joints. Appendices A3, A11, and A15 provide further details on these buildings. Damage observed in such buildings tended to be concentrated in the first story which typically was open for parking while non-structural walls provided additional stiffness on upper stories. From 52 RC buildings assessed by Japanese authorities in the Mashiki Town area, 15 were constructed prior to 1981 with 5 being deemed undamaged, 8 moderately damaged and 2 had collapsed.

JAPANESE SEISMIC DESIGN APPROACH

To understand the observed good performance of modern buildings in this strong earthquake, it is necessary to appreciate the seismic design approach used in Japan. The following section describes how seismic design forces are determined, an overview of the verification steps, and provides a comparison with NZS 1170.5 design forces.

Scientific study of earthquakes in Japan dates back to the 1880s, with building standards and guidelines only coming into effect in 1920. Seismic resistant building guidelines or detailing were not included however, and design standards and procedures were gradually developed through many significant seismic events over the decades. The first inclusion of seismic detailing in building guidelines came in 1924 following significant damage and loss of life in the Tokyo and Yokohama regions in the 1923 Kanto Earthquake. The Japan BSL was only introduced in 1950, applying to all forms of building construction. At the time, BSL did not include technical guidelines but instead referred to the previous building guidelines for such requirements. Since the introduction of BSL, many revisions and additions have been made, largely based on observed performance and damaged of buildings in major seismic events. Notable changes to BSL include major revisions in 1971 and 1981 following the 1968 Tokachi-oki and 1978 Miyagi-ken Oki Earthquakes, respectively. These revisions included changes to detailing as well as a focus on the use of larger sections and improved shear capacity of members. The procedures for the current seismic design provisions of BSL are largely unchanged after the 1981 revision. Building damage in seismic events following the 1981 revision has highlighted the improved performance of post-1981 buildings relative to the performance of pre-1971 and pre-1981 construction [5].

The seismic design forces for building design in Japan are specified in Japan BSL [6]. Two design levels are considered; Level 1 “damage limit verification” where functionality of the building must be maintained and Level 2 “safety limit verification” where life safety must be satisfied. There are no importance factors used in the determination of seismic loads in BSL, however guidelines from the Ministry of Land Infrastructure Transport and Tourism (MLIT) provide a three-level importance rating in relation to public buildings and facilities which applies an importance factor to design actions. Buildings are categorised as Normal (Importance factor = 1), School or Government Facilities (Importance factor = 1.25) and finally Hospitals and Emergency Facilities (Importance factor = 1.5).

BSL design shear force at each storey is calculated by multiplying the full live load and dead load for the storey by

Figure 7: Examples of damage to pre-1980s concrete buildings.

seismic storey shear coefficient, C_i:

$$C_i = Z R_t A_i C_o \quad (1)$$

where,

C_o is the standard shear coefficient (0.2 for Level 1 damage limit verification and 1.0 for Level 2 safety limit verification, except timber buildings use 0.3 and 1.0, respectively)

Z is the earthquake region coefficient (see Figure 8).

R_t is a value depending on the elastic period of the building and the ground conditions (see Figure 9)

A_i is a value representing the distribution of seismic shear co-efficient over the height of the building (see Figure 10) given as

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \times \frac{2T}{1+3T} \quad (2)$$

where T is the natural period of the building and α_i is the ratio of the cumulative weight from the roof to the i^{th} storey (i.e. $W_i + W_{i+1} + \ldots W_n$) to the total building weight (i.e. $W_1 + W_2 + \ldots W_n$).

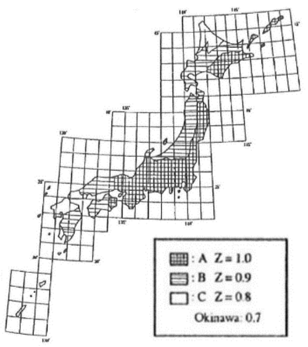


Figure 8: Z values for different locations in Japan. Figure adapted from BSL seismic loading guidelines [6].

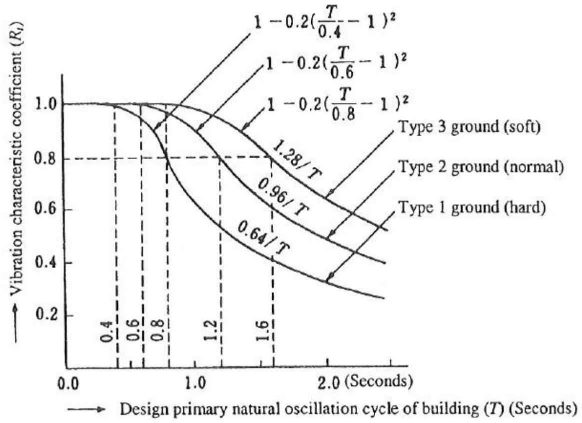


Figure 9: R_t values for buildings of different natural period founded on different soil types. Figure adapted from BSL seismic loading guidelines [6].

Note that the seismic hazard used to arrive at the design forces in the BSL is not derived from Probabilistic Seismic Hazard Analysis (PSHA) for PGA or spectral ordinates as is commonly done in most other countries including New Zealand. Instead Japan is divided into three regions (plus Okinawa) based on historical seismicity (see Figure 8), with each region assigned an earthquake region coefficient (Z) from 1.0 to 0.8 (0.7 for Okinawa only). Much of the populated east coast of the main island of Japan, Honshu, is in Zone A with Z=1.0. The majority of Kumamoto Prefecture is assigned Z=0.9 with some western regions such as Uto City assigned Z=0.8. The earthquake region coefficients have not changed in the BSL since 1979. The earthquake region coefficients, and hence the seismic design forces, vary by less than 25% across most of Japan. In New Zealand, the hazard factor (Z) varies between 0.13 for Auckland and Northland to 0.4 in Wellington and 0.6 close to the Alpine Fault at Otira and Arthurs Pass.

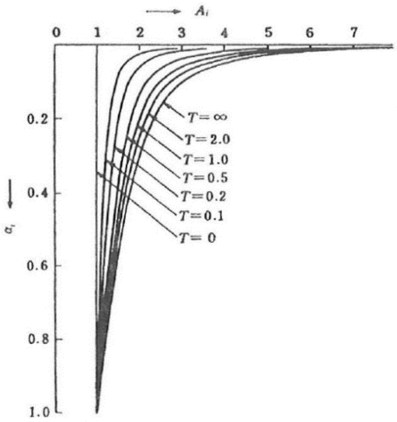


Figure 10: Sensitivity of A_i to different values of α_i. Figure adapted from BSL seismic loading guidelines [6].

It has been suggested that for Zone A (Z=1.0), the damage limit verification level is intended to represent the demand corresponding to frequent earthquakes with PGA of about 0.08-0.1g; while the safety limit verification level represents the demand corresponding to rare earthquakes with PGA 0.3-0.4g [7].

For Level 1 damage verification, drift in each storey at design shear is normally limited to a maximum of 1/200 (0.5%); but may be increased up to 1/120 (0.8%) if there is evidence of sustaining no substantial damage at this drift limit. Note the drift is an elastic drift obtained by elastic analysis without any consideration for ductility.

For buildings shorter than 31 m, the building may be deemed acceptable by checking the stiffness distribution up the building and the torsional susceptibility and ensuring that allowable stresses are not exceeded (see Table 2 (a) & (b) for allowable stress limits for RC structures).

The final step for buildings under 31 m height is to ensure sufficient strength through a simplified check of the area of columns and walls in each story. Based on the primary lateral load resisting system of the structure being frame or wall based, Equation 3 (wall) or 4 (frame) is used for the minimum wall/column area check on each floor. Note pure frame structures are then also designed to level 2 of the BSL.

$$\Sigma 2.5\alpha A_w + \Sigma 0.7\alpha A_c \geqq 0.75 Z W A_i \quad (3)$$

$$\Sigma 1.8\alpha A_w + \Sigma 1.8\alpha A_c \geqq Z W A_i \quad (4)$$

Where: $\alpha = \sqrt{\frac{f_c'}{18}}, 1 \leqq \alpha \leqq \sqrt{2}$

A_w = Plan area of shear walls in each principal direction on each floor in mm^2.

A_c = Plan area of columns on each floor in mm^2.

A_i = Vertical distribution factor as shown in Equation 2.

Z = Seismic zone factor shown in Figure 8.

(a)

Allowable Unit Stress for Sustained Forces (MPa)				Allowable Unit Stress for Temporary Forces (MPa)			
Compression	Tension	Shear	Bond	Compression	Tension	Shear	Bond
$\frac{f_c'}{3}$	$\frac{f_c'}{30}$		0.7 (0.6 when using light weight aggregate)	2 times the values of allowable unit stress for compression, tension, shear and bond for sustained forces, respectively.			
f'c represents the design concrete strength defined by BSL.							

(b)

Type of Reinforcement		Allowable Unit Stress for Sustained Forces (MPa)			Allowable Unit Stress for Temporary Forces (MPa)		
		Compression	Tension		Compression	Tension	
			Non-Shear Reo	Shear Reo		Non-Shear Reo	Shear Reo
Round		fy/1.5 (≤155)	fy/1.5 (≤155)	fy/1.5 (≤195)	fy	fy	fy (≤295)
Deformed	D ≤ 28mm	fy/1.5 (≤215)	fy/1.5 (≤215)	fy/1.5 (≤195)	fy	fy	fy (≤390)
Deformed	D > 28mm	fy/1.5 (≤195)	fy/1.5 (≤195)	fy/1.5 (≤195)	fy	fy	fy (≤390)
Where fy represents the yield stress of the reinforcement.							

W = Total weight of the building supported by the floor in consideration in N.

For Level 2 safety verification, only required when a building is over 31m or does not meet the stiffness or eccentricity ratio limits above, the design storey shear force is calculated as:

$$Q_{un} = D_s F_{es} Q_{ud} \tag{5}$$

Where

Q_{un} is the required value of ultimate lateral strength of each storey

Q_{ud} is the elastic shear force acting in each storey due to the design seismic action (i.e. product of C_i and cumulative storey weight including full live and dead loads)

F_{es} is a factor to account for the stiffness irregularity of the building in height and plan. It is calculated for each story of a building by multiplying stiffness ratio factor F_s (defined in Table 3) and the eccentricity ratio factor F_e (defined in Table 4).

Table 3: Calculation of F_s for different stiffness ratios. Table adapted from BSL seismic loading guidelines [6].

Stiffness Ratio, Rs	Stiffness Ratio Factor, Fs
Rs ≥ 0.6	1.0
Rs < 0.6	2.0-(Rs/0.6)
Where Rs represents the stiffness ratio for each story.	

Table 4: Calculation of F_e for different eccentricity ratios. Table adapted from BSL seismic loading guidelines [6].

Eccentricity Ratio, Re	Eccentricity Ratio Factor, Fe
Re ≤ 0.15	1.0
0.15 < Re < 0.3	Interpolate between 1.0 & 1.5 based on Re
Re ≥ 0.3	1.5
Where Re represents the eccentricity ratio for each story.	

The stiffness ratio, R_s, for each story must satisfy the following:

$$R_s = \frac{rs}{\bar{rs}} \geq 0.6 \tag{6}$$

where rs is the reciprocal of the relative story drift angle for each story and $\bar{rs}$ is the arithmetic mean of all the rs values for the building.

The eccentricity ratio, R_e, of each story must satisfy the following:

$$R_e = \frac{e}{re} < 0.15 \tag{7}$$

Where e is distance between the centre of mass and stiffness and re is the "elasticity radius" based on the ratio of the torsional to lateral stiffness at each story.

D_s is a value representing structural characteristics (damping and ductility) of the lateral load resisting components in each storey (see Figure 11). D_s is effectively equal to the inverse of the assumed design ductility. Figure 11 implies that the assumed design ductility for concrete frames in the Japan BSL ranges between 1.8 and 3.3; in contrast to New Zealand where the design ductility ranges from 1.25 to 6. This means that in Japanese design practice, elastic demand of regular buildings is reduced by a factor 1.8 to 3.3 depending on their ability to deform plastically, whereas in NZ the S_p and K_μ (ductility) factors can reduce the elastic demand by a factor of six or more.

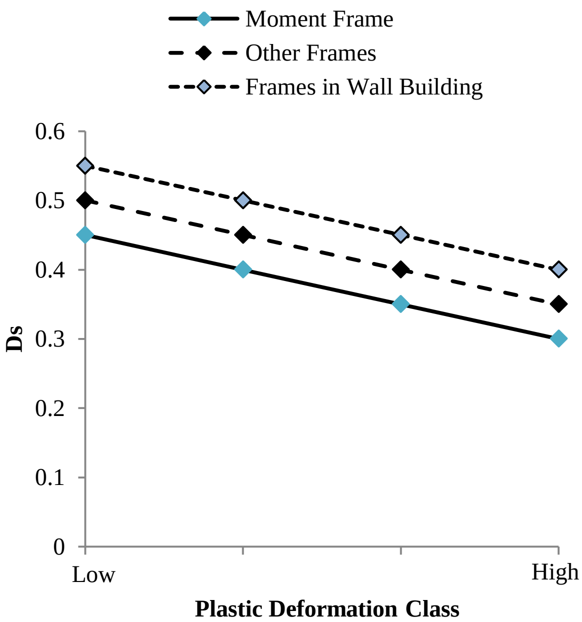
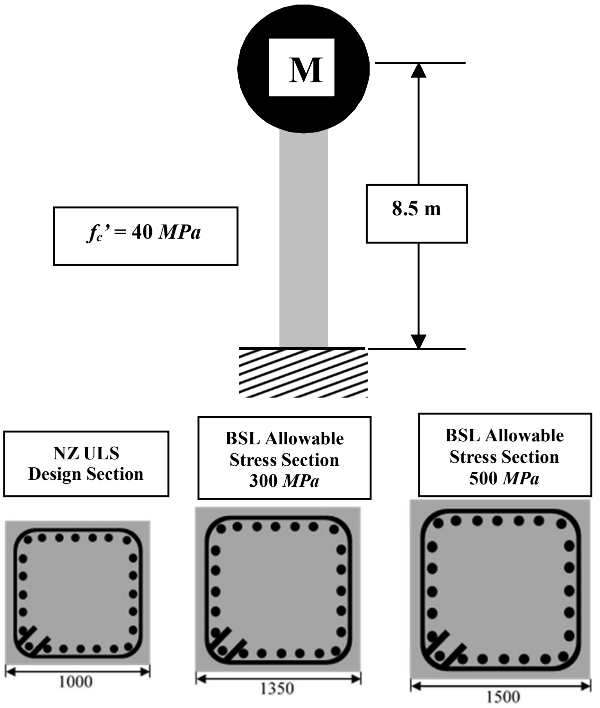


Figure 11: Example of D_s values for concrete frames. Figure adapted from BSL seismic loading guidelines [6].



	NZ ULS Design Section	BSL Allowable Stress Design Sections	
Section Size (mm)	1000 x 1000	1350 x 1350	1500 x 1500
No. of Flexural Tension Bars	15	14	18
Steel Grade (MPa)	500	500	300

Figure 12: Comprison of sections derived by New Zealand ULS vs Japanese BSL Allowable stress design for a cantilevered reinforced concrete column.

Comparison between Japanese and New Zealand Seismic Design Actions

Figure 12 outlines a simple comparison of the resultant sections from applying both New Zealand ULS design and Japanese allowable stress design to a cantilevered reinforced concrete column. It can be seen that the Japanese allowable stress design requirements do indeed result in larger and stiffer sections. This preliminary analysis shows that the Japanese stress limit design requires a section between 35 to 50% larger than that required to satisfy NZ ULS design requirements.

A summary of the key differences between the seismic design actions from the Japanese BSL and NZS 1170.5 have also been provided in Table 5. Since some differences can be counteracting, it is useful to compare the story shear forces for an example 10-story building located in different cities in New Zealand and Japan as shown in Figures 13 to 15. This comparison makes it clear that the design actions in Kumamoto are greater than most locations in New Zealand, particularly for the safety (Level 2) check, and exceed ULS design forces in Christchurch by approximately a factor of 2 (depending on soil type assumed).

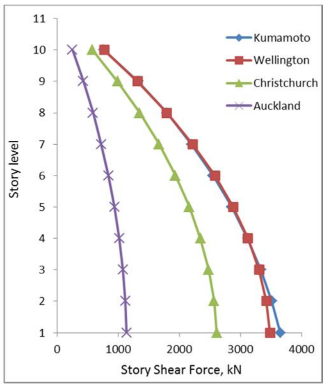


Figure 13: Story Shear Force for Kumamoto (Damage Check, Soil Type 3) vs NZS1170 SLS (Ductility 1.25, Soil Type E).

Note: If ductility of 1 is used, the three NZ plots will increase by 25%. For Tokyo, the demand will be about 11% higher than Kumamoto. If Soil Type D (arguably within the range covered by Japan soil type 3) is used for NZ, the NZ demand will reduce by 35%.

Table 5: Comparison of key differences between NZS 1170.5 and Japan BSL.

	NZS 1170.5	BSL (H<31m)	BSL (H>31m)
Seismic Hazard	Based on PSHA using return period of 500 years for ULS.	Based on historical seismicity leading to identification of three zones. No specific return period considered.	
Soil Types	Five soil types with approx. max amplification from rock at short and long periods of 1.5 and 3.0, respectively.	Three soil types with no application at short periods and approx. max amplification of 2.0 at long periods.	
Seismic Weight	Dead load plus reduced (0.6 or 0.3) live load.	Dead plus full live load (Note, live load specified in Japanese code for calculation of seismic mass is significantly lower.	
Serviceability (Level 1 Damage limit)	Drift limits based on separation of components.	Drift limits: 1/200 (0.5%); but may be increased up to 1/120 (0.8%).	
	Strength check (does not typically govern).	Allowable stress checks ï See Table 2 for limits.	
ULS (Safety Limit)	Drift limit = 2.5% (Elastic drifts are amplified by kdm = 1.2 to 1.5 and by design ductility).	No ULS design unless stiffness and torsion requirements are not satisfied. Lateral capacity check required based on total cross sectional area of walls and columns as well as shear design.	None specified.
	Strength check using ductility factors ranging from 6 to 1.25 (additional reduction by Sp=0.7).		Strength check using ductility factors ranging from 3.3 to 1.8.
Irregularity Checks	Irregularity checks limit types of analysis and ductility of structures.	Hard limits on: -Vertical stiffness distribution -torsional susceptibility (If not satisfied must do ULS check for H>31m).	

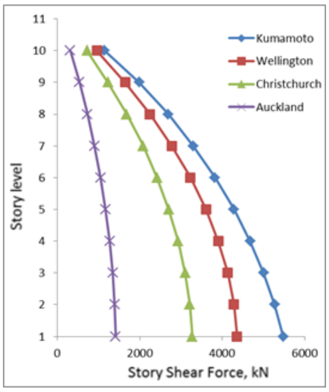


Figure 14: Story Shear Force for Kumamoto (Safety Check, Soil Type 3, D_s=0.3) vs NZS1170 ULS (IL2, Ductility 4, Soil Type E).

Note: D_s=0.3 is assigned for the most ductile category allowed in Japanese code. For Tokyo, the demand will be about 11% higher than Kumamoto. If Soil Type D (arguably within the range covered by Japan soil type 3) is used for NZ, the NZ demand will reduce by 35%.

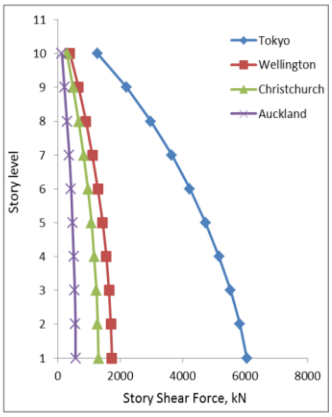


Figure 15: Story Shear Force for Tokyo (Safety Check, Soil Type 2, D_s=0.3) vs NZS1170 ULS (IL2, Ductility 4, Soil Type C).

Note: D_s=0.3 is assigned for the most ductile category in BSL. For important buildings, the NZ demand will increase by 30% for IL3 and 80% for IL4 (might be larger for IL4 as SLS2 criteria kicks in and ductility of 4 might not be tenable). No change for Japan. Even with 80% increase for an emergency building in Wellington, the building designed in Tokyo will still be designed for a base shear demand that is almost twice as that for IL4 building in Wellington.

Noting that the drift limits are applied for Level 1 in BSL and for ULS in NZS 1170.5, it is of interest to compare the ratio of the drift limits and the elastic design forces as shown in Table 6 for the example 10 story building considered in Figures 13 to 15. The ratios shown in Table 6 suggest that the drift limit check at ULS for NZS1170.5 is approximately equivalent to the drift limit check at Level 1 in the Japan BSL and thus this design parameter cannot be attributed to the overall stiffer structures which were observed in Japan.

Table 6: Comparison of drift limits in NZS 1170.5 and Japan BSL.

	Drift limits (from elastic analysis)	Elastic design shear (from Fig 13)
ULS NZS 1170.5 (Christchurch)	1.7 to 2.1% (= 2.5% / 1.5 to 1.2)	12800 kN
BSL Damage limit (Kumamoto)	0.5% to 0.8%	3700 kN
Ratio	2.1 to 4.2	3.5

Some further comparisons of design actions derived from BSL and NZS 1170.5 are shown in Table 7. It can be seen that seismic actions for design scenarios in Kumamoto, comparable to conditions in Christchurch, yield reduced base shears of up to 3.6 times larger than those from NZS 1170.5.

Table 7: Design base shear coefficient for different design scenarios in NZ and Japan. Note all NZS 1170.5 actions are for 50-year design life and Importance level 2. Hazard and soil conditions were selected for comparison between typical soft soil conditions in Christchurch and Kumamoto.

NZS 1170.5						
Height, m	14	14	35	35	60	60
	SLS	ULS	SLS	ULS	SLS	ULS
Weight, KN	7920	7920	20400	20400	41200	41200
Soil Type	Soft, D	Soft, D	Soft, D	Soft, D	Soft, D	Soft, D
Period, s	0.5	0.5	1	1	1.6	1.6
$C_h(T)$	3	3	1.93	1.93	1.358	1.358
Z Factor	0.3	0.3	0.3	0.3	0.3	0.3
R Factor	0.25	1	0.25	1	0.25	1
Ductility	1.25	4	1.25	4	1.25	4
S_p Factor	0.7	0.7	0.7	0.7	0.7	0.7
K_μ Factor	1.18	3.14	1.25	4	1.25	4
Design C_d	0.133	0.201	0.081	0.101	0.057	0.071

Japan Building Standard Law						
Height, m	14	14	35	35	60	60
	Level 1	Level 2	Level 1	Level 2	Level 1	Level 2
Weight, KN	7920	7920	20400	20400	41200	41200
Soil Type	Soft 3	Soft 3	Soft 3	Soft 3	Soft 3	Soft 3
Period, s	0.28	0.28	0.7	0.7	1.2	1.2
R_t Factor	1	1	1	1	0.95	0.95
Z Factor	0.9	0.9	0.9	0.9	0.9	0.9
C_o Factor	0.2	1	0.2	1	0.2	1
Ductility Factor, D_s	N/A	0.3	N/A	0.3	N/A	0.3
Irregularit Factor, F_{es}	N/A	1	N/A	1	N/A	1
Base Shear, KN	1425.6	7128	3672	18360	7045.2	35226
Elastic Design C_d	0.180	0.900	0.180	0.900	0.171	0.855
Reduced Design C_d	N/A	0.270	N/A	0.270	N/A	0.257

JAPANESE SEISMIC EVALUATION OF EXISTING REINFORCED CONCRETE BUILDINGS

Seismic evaluation of pre-1981 concrete buildings is done in Japan according to the *Japanese Building Disaster Prevention Association (JBDPA) Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings* [8]. Similar to many other assessment guidelines and standards internationally (e.g. ASCE 41 [9], Eurocode 8 Part 3 [10], and draft NZ guideline *Seismic Assessment of Existing Buildings* [11]), JBDPA provides a means of assessing and retrofitting buildings designed prior to the development of current seismic design codes. The standard calculates a seismic capacity index, I_S based on the product of the strength and ductility for each story of the structure and compares this coefficient to a seismic demand index, I_{So}:

$$I_S \geq I_{So} \tag{8}$$

The seismic capacity index, I_S is the minimum value of ${}_iI_S$ (Equation 9) determined on a floor-by-floor basis in each principal direction of the building. It is dependent on the strength and ductility of structural members with modifiers for age and shape of the building.

$${}_iI_S = {}_iE_0 . S_D . T \tag{9}$$

Where, i = signifies the floor number, S_D = irregularity index related to the shape of the building, T = aging index and E_0 = seismic capacity index.

The seismic capacity index, E_0 is dependent on the ductility and strength of the structural elements on each floor, and hence, a representation of the energy dissipation capacity of these elements. E_0 is calculated based on Equation 10:

$$ {}_iE_0 = C_i . F_i . \phi \tag{10} $$

where, C_i = Strength index at ith floor, F_i = Ductility index at ith floor and ϕ = Story shear distribution index estimated simply as $\frac{n+1}{n+i}$, where n is the number of stories in the building.

The seismic demand index, I_{So} is determined as per Equation 11:

$$ I_{So} = E_s . Z . G . U \tag{11} $$

where, E_s = Basic Seismic Demand index equal to 0.8 for level 1 screening, 0.6 for level 2 and 3 screening, Z = Zone index from Figure 8, G = Ground index, accounting for amplification effects of the surface soil and U = Importance Factor as described in the previous section on the Japanese BSL.

As previously alluded to, the JBDPA evaluation guide provides three different procedures for the calculation of the strength (C) and ductility (F) indices. The three levels increase in complexity and accuracy, such that an engineer will typically start with Level 1 and only proceed to Level 2 or 3 if requirements are not satisfied. These procedures are briefly described below but further details of all three levels can be found in Section 3 of the JBDPA *Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings*.

Level One

Simplest method which may be considered analogous to a screening method or the Initial Evaluation Procedure (IEP) in New Zealand.

Strength Index, C: Calculated based on the cross-sectional area of walls and columns on each floor.

Ductility Index, F: For this level the vertical elements are categorized into two groups, walls and columns (F = 1.0), and short columns (F = 0.8).

Level Two

This method assumes a story mechanism as this is the most common mechanism observed in damaged buildings in Japan.

Strength Index, C: C is calculated from the ultimate lateral load carrying capacity of vertical members (columns and walls) with the assumption that beams are “strong enough” to force a strong-beam-weak-column mechanism. The strength index calculation in this level, considers the failure mode of the vertical load carrying elements i.e. shear/flexure controlled columns and walls, as well as brittle columns. The index is calculated based on the minimum of the ultimate shear strength, Q_{su} and the shear force Q_{mu} at the development of a flexural mechanism. Where Q_{su} is the lower of the two, the member is classified as shear-controlled and where Q_{mu} is the lower the member is classified as flexure-controlled.

Ductility Index, F: The Guideline provides a set of equations for the determination of the ductility index at each floor based on the classification of the vertical elements as shear or flexure controlled from the calculation of the C index.

Level Three

This is similar to the procedure in Level Two except that the collapse mechanism of the building is determined from analysis rather than assuming a story mechanism.

Strength Index, C: The effect of earthquake induced axial loads are now taken into consideration in the calculation of the ultimate load carrying capacities. A detailed analysis is used to determine the location of hinges in the structure based on a comparison of beam and column yield moments at joints.

Ductility Index, F: A set of detailed equations are provided for the determination of the ductility index, in accordance with the vertical member classification provided in the Level Three procedure. The classification system includes categories for flexure-controlled and shear-controlled beams and walls with uplift.

Since S_D, T, ϕ, Z, G, U are frequently equal to 1.0, the methodology effectively simplifies down to:

$$ Is = C\,F > 0.6\;(or\;0.8\;for\;Level\;1) \tag{12} $$

Note that a structure with an I_S index above 0.6 is generally expected to exhibit good performance based on historical evidence of past damaging earthquakes (see Figure 16). This empirical limit has been supported again by the observed damage to buildings in the Kumamoto earthquake (where available, *Is* values are provided in Appendices for buildings inspected).

While the *Is* limit can be achieved through increased strength (C) or ductility (F), emphasis has recently been placed on increases in strength (C) if post-earthquake functionality is a desired performance objective in the retrofit design. Increasing ductility is only seen as a viable option if focus of retrofit is solely for life safety.

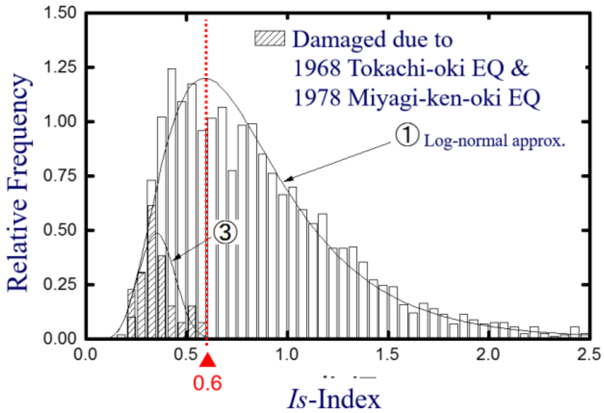


Figure 16: Statistics of Is index values of public RC structures in comparison to damaged RC structures in Shizuoka, Japan – Figure adapted from Nakano & Teshigawara [12].

JAPANESE POST-EARTHQUAKE DAMAGE EVALUATION PROCEDURE

Following major damaging earthquakes in Japan, a rapid inspection process is undertaken where, similar to New Zealand, buildings are classified into one of three categories and Red, Yellow, or Green placards are placed on the outside of buildings, clearly identifying the level of damage.

It should be noted that in contrast to New Zealand, placards in Japan are only advisory, hence owners and tenants could choose to enter and occupy yellow and red placard buildings. Occupancy of damaged buildings was observed by the reconnaissance team.

Building placards can be removed or changed following the rapid assessment, if the identified issues or risks are addressed. Private building owners carry the responsibility for ensuring that repairs or reconstruction takes place, while public buildings will be systematically addressed by regional government. A varying level of funding is also made available by the national government for both private and public buildings for repairs and reconstruction, depending on yearly budgets and damage sustained following the earthquake.

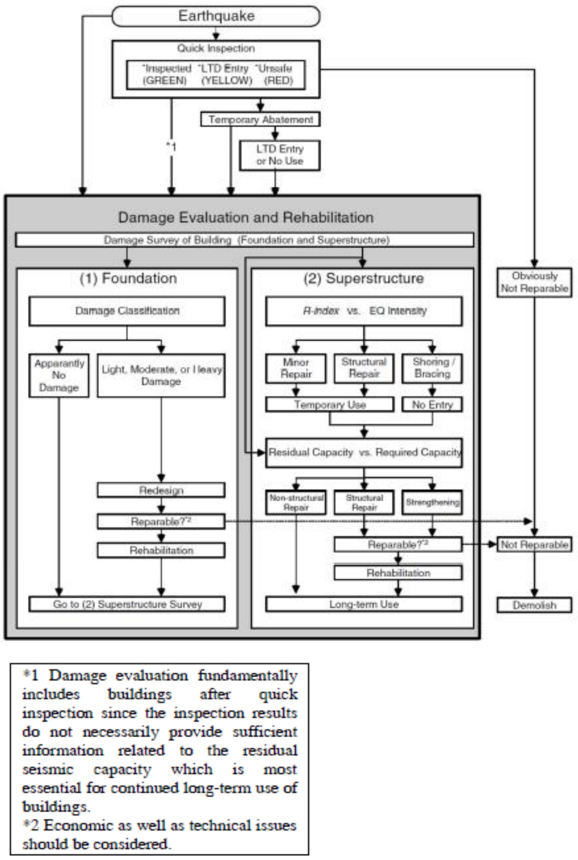


Figure 17: Flow diagram summary of Japanese Damage Evaluation and Rehabilitation Guideline. Adapted from Maeda et al. 2014 [12].

Figure 17 shows the building assessment process following an earthquake. Following the rapid assessment procedure, a more detailed damage assessment is then carried out. In Japan, the residual seismic capacity of buildings is currently defined by the R-index in accordance with the Japanese Damage Evaluation Guideline (JDEG). The guideline, established in 1991, was originally based on the evaluation of buildings with a story-collapse failure mechanism, being the most common failure mechanism in Japanese RC buildings. A revision of the guideline was released in 2015, with the inclusion of an R-index evaluation method for buildings where a full frame mechanism develops with yielding of beams throughout the structure.

The general calculation for the residual seismic capacity ratio, R-index, is shown in Equation 13:

$$R = \frac{{}_{D}I_S}{I_S}.100 \qquad (13)$$

Where, I_S = original seismic capacity index discussed previously and ${}_{D}I_S$ = post-earthquake seismic capacity discussed below.

Similar to the I_S index, the ${}_{D}I_S$ index (Equation 14) is also determined based on the strength (C) and ductility (F) indices. A seismic capacity reduction factor, ƞ is also applied to these indices when calculating ${}_{D}I_S$. The reduction factor take into consideration the deterioration in the energy dissipation capacity of each structural element, represented by the product of the strength and ductility indices. ${}_{D}I_S$ is determined on a floor by floor basis in each principal direction.

$${}_{D}I_S = \Sigma(\eta.C.F).S_D.T \qquad (14)$$

The reduction factor ƞ, are determined for all structural elements on a floor to calculate the ${}_{D}I_S$ index. In practice these reduction factors are determined through inspection of the structural elements and classification of their damage state as described by the JDEG guidelines. For RC Buildings, JDEG defines five levels of damage for both ductile and brittle elements which are used to classify individual structural elements (Table 8 & 9).

Table 8: JDEG Damage Level Classification for ductile vertical elements. Adapted from Maeda et al. 2014 [13].

Damage Class	Observed Damage on Structural Members
I	Some cracks found. Crack widths smaller than 0.2mm.
II	Cracks widths of 0.2 - 1mm are found.
III	Some heavy cracks of 1 – 2mm wide are found. Some concrete spalling is observed.
IV	Many heavy cracks are found. Crack widths larger than 2mm. Reinforcing bars exposed due to spalling of cover concrete.
V	Buckling of reinforcement, crushing of concrete core, vertical deformation of columns and/or shear walls. Subsidence of upper floor and/or fracture of reinforcing bars are observed in some cases.

Table 9: JDEG Damage Level Classification for brittle vertical elements. Adapted from Maeda et al. 2014 [13].

Damage Class	Observed Damage on Structural Members
I	Some cracks found. Crack widths smaller than 0.2mm. Diagonal X crack patterns may be visible.
II	Cracks widths of 0.2 - 1mm are found. Diagonal X crack patterns may be visible.
III	Some heavy cracks of 1 – 2mm wide are found. Some concrete spalling is observed. Diagonal X crack patterns may be visible.
IV	Many heavy cracks are found. Crack widths larger than 2mm. Reinforcing bars exposed due to spalling of cover concrete. A significant reduction in both lateral and vertical load carrying capacity of vertical elements.
V	Buckling of reinforcement, crushing of concrete core, vertical deformation of columns and/or shear walls. Subsidence of upper floor and/or fracture of reinforcing bars are observed in some cases. Widening of X-shape shear cracks and sudden loss of lateral and vertical load carrying capacity of structural elements.

Each class is defined with observable characteristics of the sustained damage with key factors used to differentiate ductile and brittle elements such as distinct crack patterns. In the latest edition of JDEG, elements are further classified into ductile, quasi-ductile and brittle columns, shear walls and beams. As described by Maeda et al. [13], the ƞ factors in the guideline were determined based on experimental data where ƞ is calculated based on the residual energy dissipation capacity of a structural element as shown in Figure 18.

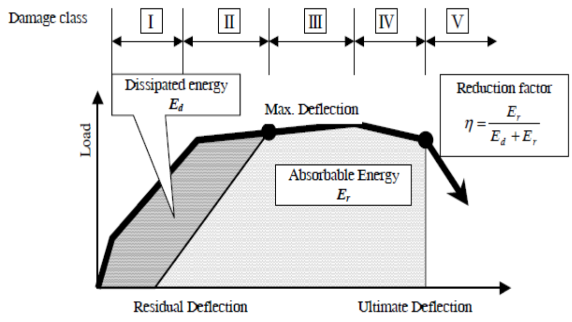


Figure 18: Seismic Capacity Reduction Factor. Adapted from Maeda et al. [13].

The resultant R-index which is obtained from application of JDEG can be used to classify the building into the following categories shown in Table 10.

Table 10: JDEG R-index Building Damage Classification. Adapted from Maeda et al. 2014 [13].

Damage Level	Residual Capacity
Slight Damage	R ≥ 95%
Minor Damage	80 ≤ R < 95%
Moderate Damage	60 ≤ R < 80%
Severe Damage	R < 60%
Collapse	R = 0%

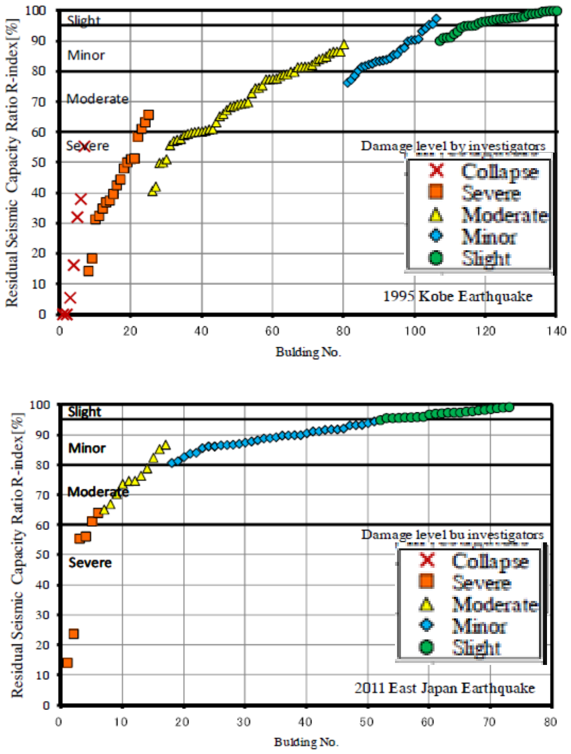


Figure 19: R-index evaluation results vs Engineer's expert judgment for RC school buildings damaged in 1995 Kobe Earthquake and 2011 East Japan Earthquake. Figure adapted from Maeda et al. [13].

The 95% threshold has been set to correspond with the serviceability level performance of buildings and hence buildings with 95% or above R-index do not require any rehabilitation. JDEG provides guidance based on the intensity of the earthquake, R-index of the superstructure and foundation, the cases which required rehabilitation or in severe cases detailed analysis for further options and finally demolition. Maeda et al. (2014) outlines the results of the application of JDEG to 70 RC school buildings following the 2011 East Japan Earthquake and 140 RC school buildings following the 1995 Kobe Earthquake. The results of this analysis have been plotted against the judgment of experts based on their observations of these buildings. This plot is shown in Figure 19 and the close correlation between the R-index boundaries shown in Table 10 and expert's judgment is evident.

CONCLUDING OBSERVATIONS

The April 2016 Kumamoto earthquakes share similarities with the 2010-11 Canterbury Earthquake Sequence; both comprised of multiple damaging events, including a shallow event in close proximity to a major urban center and similar levels of shaking for structures with periods above 0.7s. These similarities provide a unique opportunity for comparison of building performance in the two countries and possible lessons for structural design.

The observations from Kumamoto particularly highlighted a difference in the resilience of New Zealand and Japanese buildings and the subsequent impact on the city as a whole. The overarching impression in Kumamoto was of a functioning city with the majority of the population being able to carry on with their daily lives with minimal disruption, and no sight of closed off areas or "Red Zones" two months after the earthquake. In contrast, an exclusion zone was maintained in Christchurch CBD for over 2 years following the February 2011 earthquake.

Modern code designed structures in Kumamoto sustained limited damage and exhibited good performance on the whole. The inspected buildings pointed to a design philosophy emphasising stiffer and stronger buildings than those commonly seen in New Zealand and few examples of high ductility demands were noted. Observations of stiffer and stronger structures have been supported by the analysis of the Japanese BSL seismic design guidelines. As outlined in this report, Japanese structures are designed to higher base shears and assume ductility of 1.8 to 3.3, compared to design ductility of 1.25 to 6.0 in New Zealand. The Japanese BSL adopts a simple allowable stress design approach for the Level 1 design, typically resulting in larger and stiffer sections than New Zealand ULS design. On the other hand, the BSL does not focus on the identification of ductile mechanisms and capacity design to the same extent as New Zealand Standards.

It is also worth noting that the seismic design forces in the BSL have not changed since 1981, while NZS design forces change frequently as the science for determining probabilistic seismic hazard evolves. While many codes in the world have "scientifically progressed" from seismic zones to probabilistic assessment of hazard, the Japanese BSL has remained far more stable by avoiding the delusional precision of estimating future earthquake demands and the reliance on a constantly changing science in setting building design forces.

Non-structural elements were also observed to have performed favorably in the Kumamoto earthquake. Proactive practices such as passive fire resistance and removal of suspended ceiling panels were also observed in order to minimise internal damage due to nonstructural systems and allow offices and schools to return to a functioning state as rapidly as possible.

The implementation of post-earthquake rapid assessment was also observed in Kumamoto, using a tagging procedure similar to the system used in New Zealand. Unlike the New Zealand system, however, the building tags are only advisory in nature, and although they are largely adhered to by the public they are not strictly enforced. There was also an emphasis in damaged areas to maintain community services in place with government and school facilities being kept on site through the use of temporary offices and classrooms.

Overall, the observations of the performance and community response in Kumamoto highlighted similar goals to New Zealand where life safety was concerned; however, in terms of resilience and return to function, the observations were quite contrasting to what was experienced in New Zealand following both the Christchurch and Kaikoura Earthquakes. This contrast suggests that the New Zealand structural engineering profession should carefully consider if the widely accepted design philosophy relying on high ductility response of modern structures could be replaced by a focus on strength and stiffness leading to lower ductility demands and faster recovery after earthquakes.

Research is required to identify the cost of such changes to building designs in New Zealand and potential architectural impacts due to the need for larger structural elements. Cost considerations must not only include higher up-front costs, but also savings in terms of life cycle costs due to lower damage in future earthquakes.

The authors encourage the continued exchange and comparison of design philosophies and field experiences between the earthquake engineering professions of Japan and New Zealand.

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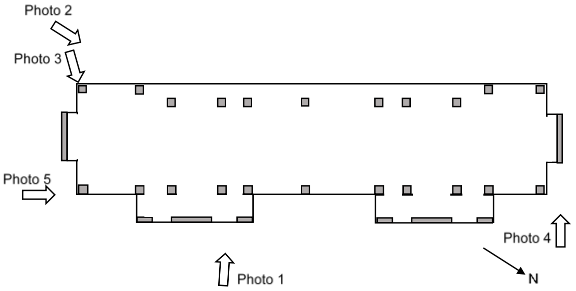
APPENDIX A

BUILDING DAMAGE REPORTS

Building Damage Report - A1

Item	Details
Street Address/Location	Apartment C: Irichimachi, Uto City
Construction Year	1998
Building Description	Private residential apartments. Structural form is regular in plan with pilotis ground floor.
Building Footprint / Floor Area	48x12m
No. of storeys / basements	4
Structural system	Regular RC moment-frames with flat-slab floor system. Transverse end walls, and walls around the stairs. Infill walls non-structural, insitu RC stairs.
Earthquake resisting system	RC MRFs + end walls in transverse direction. Some contribution likely from the walls associated to the stairs although uncertain if intended.
Foundation system	Uncertain but possibly shallow footings due to spreading of wall bases away from the footprint.
Past seismic strengthening	None
Likely Design Standards	Latest Japanese Design Code (post 1981)



Plan showing floor slab outline and full-height structural elements and photo locations.

Photo 3: Column hinge and end wall footing spread

Photo 1: North-east part elevation.

Photo 2: South-west part elevation.

Photo 4: Plastic hinge at top of column.

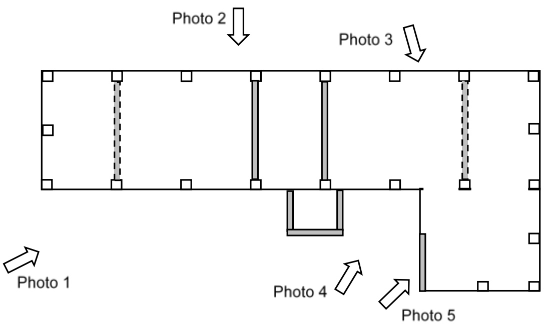
Photo 5: Column plastic hinge with intact concrete core.

Observations

- The damage observed from the outside of this building was limited to the ground floor only. All of the columns had formed plastic hinges at the top of the columns and very limited damage was observed at the column bases. Possibly there is column damage below ground level.
- The column plastic hinges may be due to capacity design not being followed through completely with the interaction of the first floor transfer slab. There appeared to be bars terminated within the top of the ground floor columns, however it is uncertain if these are column bars or 90 degree bends from slab and beam steel. Generally the hinge was well confined, with the concrete core remaining intact, although vertical bar buckling was observed in many locations.
- The end transverse walls had formed horizontal cracks as a result of the wall folding against the Level 1 transfer slab. There was also noticeable spread of the wall base away from the building, indicating that the footings were not tied into the rest of the building foundation system.
- The walls around the stairs had limited damage. Given the short MRF bays in the longitudinal direction is likely that these were not intended to act as seismic resistance. Details of the slab connections could not be observed to understand if the stair areas were effectively isolated from the main building.

Building Damage Report – A2

Item	Details
Street Address/Location	Yasuyomachi 558-1. Chuo District, Kumamoto City
Construction Year	1991
Building Description	Private residential apartments
Building Footprint / Floor Area	50x12m + 15x10m wing towards river
No. of storeys / basements	11
Structural system	RC moment-frames with flat-slab floor system with transverse walls and walls around service cores. Insitu RC stairs. Non-structural infill walls connected to MRF beams
Earthquake resisting system	RC MRFs + walls in transverse direction
Foundation system	Uncertain. Presumably piles
Other notable features	Over central bike store/walk-through a ‘non-structural’ infill panel transfers out at first suspended floor level.
Past seismic strengthening	None
Likely Design Standards	Designed to Current Code approach



Plan showing floor slab outline with known full-height structural elements and photo locations.

Photo 1: North-west elevation.

Photo 2: East part elevation.

Photo 3: Typical infill concrete wall shear cracking due to interaction with moment frame beams.

Photo 4: Moment frame beam flexural hinging in regions not affected by infill walls.

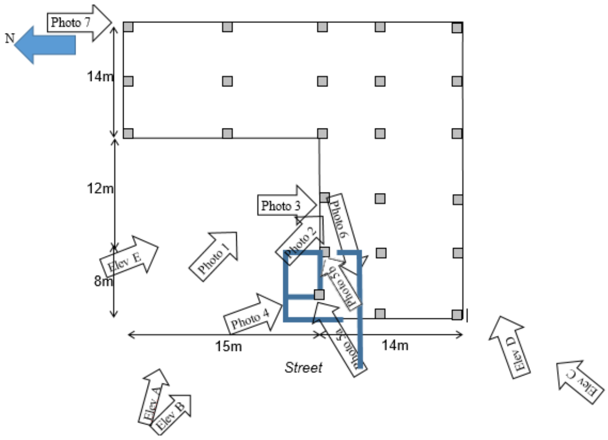
Photo 5: Foundation settlement at base of shear walls due to rocking.

Observations

- Well distributed flexural cracks in some of the moment-frame beams, but no cracking in the columns or joint regions. This would suggest that the strong-column weak-beam design intent for the moment-frames was achieved where the infill walls didn't interfere with the frame response.
- Some damage observed to the pavement around the core-walls on the north-west side of the building and settlement relative to the surrounding pavement. This suggests that the walls (particularly those attached the perimeter of the building) have rocked with limited soil failure. The foundation system is unknown but if these cores were not considered part of the primary lateral-force resisting system then they may not have the same foundation treatment as the rest of the building.
- Within each moment-frame bay, 'non-structural' reinforced concrete infill walls were used to define windows and doors, however these were constructed to be integral with the moment-frames. Extensive shear cracking was observed in these elements.
- The observed damage in this building represented typical reinforced concrete design and construction practice that is understood to have been applied in Japanese seismic design for a number of years under previous design codes, and to some extent under the current code. The rigidly connected 'non-structural' panel approach has only recently has been phased out of standard practice.

Building Damage Report – A3

Item	Details
Street Address/Location	Demachi 1-5, Nishi District, Kumamoto City, north of Kumamoto Castle
Construction Year	1975
Building Description	Apartments
Building Footprint / Floor Area	~700m^2
No. of storeys / basements	7 stories (no basement)
Structural system	RC moment frame. No apparent structural walls in building.
Earthquake resisting system	RC moment frame with pilotis ground floor for parking
Foundation system	Unknown
Past seismic strengthening	None
Likely Design Standards	Pre-1981 code



Plan showing building layout with known full-height structural elements and photo locations

(note walls shown in blue were not clearly structural walls)

Elevation C

Elevation A

Elevation D

Elevation B

Elevation E

Elevation F

(note complete collapse of first story except near elevator core along street)

Photo 1

Photo 2

Note: base of column shear failure and joint failure (potentially induced as column pushed out by failure plane)

Photo 3

(smooth 10mm ties @ 100; no joint transverse reinforcement)

Photo 4: Uplift of wall (does not appear to be structural wall).

Photo 5a: crushed columns at corners of walls around core.

Photo 5b: crushed columns at corners of walls around core.

Photo 6: Damage to first floor diaphragm at wall (similar damage not apparent on other levels).

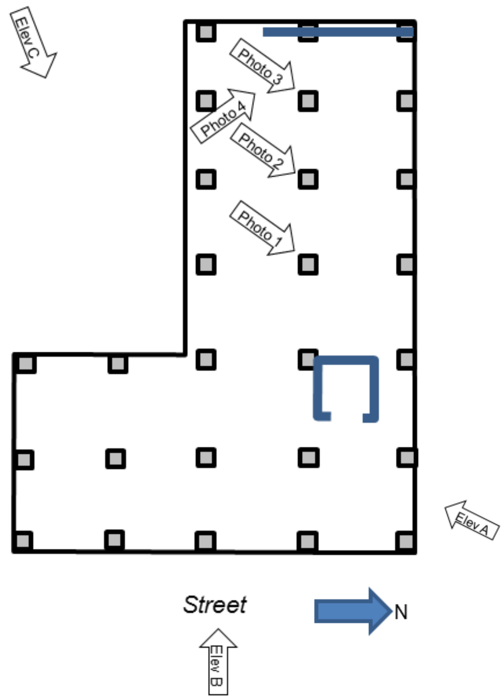
Photo 7

Observations

- Complete collapse of first story except along street.
- RC walls around elevator and stairs not structural (limited anchorage to foundation), but may have contributed to torsional response of building.
- No apparent structural walls in building.
- Most columns crushed and unable to identify mode of failure.
- Photos 1-3 show column with shear failure at base and joint failure – expect shear failure occurred first followed by joint failure when column slides down shear failure plane and is forced outward.
- Photo 6 shows damage to diaphragm near core (possible shear damage), but similar damage not observed at upper levels and thus believed to be damage induced by bending (or compression) of slab during collapse.
- Two similar buildings located next door and across the street did not collapse (Refer building summary A4 for description of building across the street).

Building Damage Report – A4

Item	Details
Street Address/Location	Nishi District, Kumamoto City, north of Kumamoto Castle
Date of construction	1970
Description / Building Occupancy	Apartments
Building Footprint / Floor Area	~700m^2
No. of storeys / basements	7 stories (no basement)
Structural system	RC moment frame
Earthquake resisting system	RC moment frame with walls, pilotis ground floor for parking
Foundation system	unknown
Stair System	unknown
Past seismic strengthening	none
Likely Design Standards	Pre-1981 code
Other	Across street from building described in Appendix A3 (collapsed)



Plan showing building layout with known full-height structural elements and photo locations (walls shown in blue – uncertain if core is structural)

Elevation C

Elevation A

Elevation B

Photo 1: Bidirectional shear cracks.

Photo 2: Bidirectional shear cracks.

Photo 3b: Apparent torsional cracks.

Photo 3a: Apparent torsional cracks.

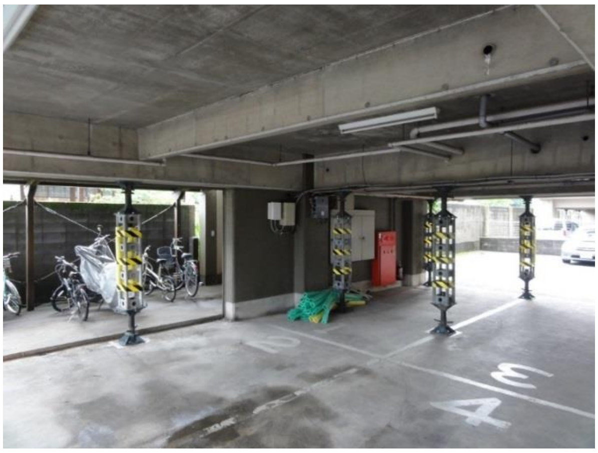


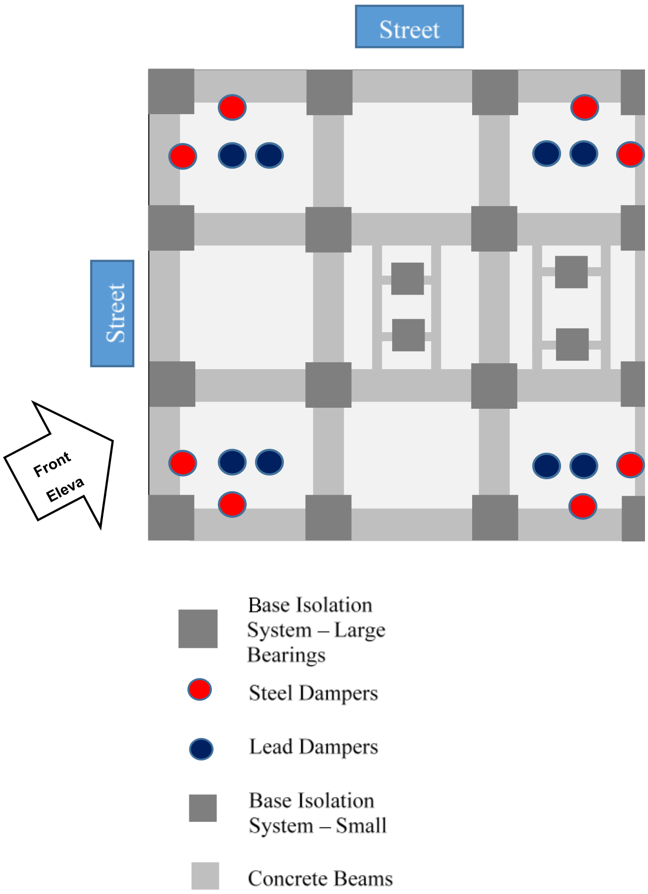
Photo 4: Wall on west end of building.

Observations

- Across street from collapsed building. (Appendix A3)
- Built by same owner 5 years prior to collapsed building.
- Both designed to pre-1981 code
- Primary difference is presence of wall at west end of building and location of core in the corner of L-shape (could not confirm if core is structural)
- Significant bidirectional shear cracking on two interior columns – bidirectional shear appears more prevalent in Japan due to similar size beams in two orthogonal directions.
- Cracking on third column appears torsional (cracks wrap around column

Building Damage Report – A5

Item	Details
Street Address/Location	Kumamoto City, Kumamoto Prefecture
Date of construction	Approx 2010
Building Description	Private Residential Apartments, Structural form is regular in plan
Building Footprint / Floor Area	~29m x29m (~850 m^{2})
No. of storeys / basements	15 stories (+ 1 Storey Basement)
Structural system	RC moment frame in orthogonal directions
Earthquake resisting system	RC moment frame with base isolation at underside of ground floor slab level in basement
Foundation system	Foundation below base isolation unknown
Other notable features	Features rubber bearings with a combination of both steel and lead dampers
Past seismic strengthening	none
Likely Design Standards	Latest Japanese Design Code (post 1981)
Placard and date (if known)	Green, building fully occupied



Sketch plan of building layout showing isolation system layout

Building street elevation

Photo 1: Lead dampers.

Photo 2: Steel dampers.

Photos 3 & 4: Separation of steel dampers from concrete beams.

Photo 5: Failure at joint between floor slab & lead dampers.

Photo 6: Failure at joint between floor slab & lead dampers.

Photo 7: Failure at joint between floor slab & lead dampers.

Photo 8: Small base isolator (see floor plan for location).

Photo 9: Visible signs of cone failure like cracking in base slab where small base isolator is connected.

Photo 10: Cracking in deck slab where connected to small base isolator

Observations

- Damage concentrated in basement area and ground floor concrete slab where the steel and lead dampers are connected to the building structure.
- No serious damage to main base isolation system. Majority of damage to damping system.
- Steel dampers were not positioned to fully align with the structural beams above but are significantly eccentric in relation to the main structure beams. The stub columns above the steel dampers show damage caused by structural separation of these columns from the main building structure (refer Photos 3 and 4).
- Lead dampers were all located away from the main structure beams and the stub columns above these dampers were connected to the ground floor slab above only.
- All lead dampers showed failure at connection to the ground floor slab (refer Photos 5-7). Limited anchorage of longitudinal reinforcement observed.
- Significant damage observed at the junction of ground floor slab in the area of the dampers stub columns.
- Structure consists of 16 base isolated concrete columns supported on laminated elastomeric bearings, with four additional small bearings connected to the base slab (No columns above).
- Some cracking was observed above small base isolation columns. This cracking was observed through the base slab in areas where there was access to inspect (refer Photos 9 and 10)

Building Damage Report – A6

Item	Details
Street Address/Location	Two similar adjacent buildings Kumamoto City, Kumamoto Prefecture
Date of construction	Unknown
Building Description	Privately owned Apartments
Building Footprint / Floor Area	Unknown
No. of storeys / basements	Each building is 15 stories (+ 1 Storey Basement)
Structural system	RC moment frame
Earthquake resisting system	RC moment frame with base isolation at underside of ground floor level in one building and a mixture of ground and first floor for the other building
Foundation system	Foundation below base isolation unknown
Other notable features	Stairs and Elevators Base Isolated on separate floor to rest of structure on Building No.2.
Past seismic strengthening	none
Likely Design Standards	Latest Japanese Design Code (post 1981)
Placard and date (if known)	Green, building fully occupied

Building front elevation for Building No.3

Photo 1: Evidence of residual displacement at base of building

Photo 2: Damage to the first level of stairs on Building No.2 with multi-level isolation

Photo 3: Damage to the first level of stairs on base isolated Building No.2. Staircase isolated at ground level while isolation plane of main structure was at level 1.

Photo 4: Damage to the first level of stairs on base isolated Building No.2. Staircase isolated at ground level while isolation plane of main structure was at level 1.

Observations

- Both buildings exhibited approximately 300mm of residual displacement. Their design allowed for up to 600mm
- One building has a multilevel base isolation system with the main structure isolated a first floor while the stairs and elevator shaft extend to ground with an isolation (sliding) joint at ground floor.
- This building showed significant cracking and damage to the concrete wall and associated elements to one of the staircases between ground floor and first floor. It appears the base of the stair did not slide as the design had intended.
- The other building is isolated completely at ground floor on single concrete raft. No visible damage was observed.

Building Damage Report – A7

Item	Details
Street Address/Location	Kumamoto Prefecture
Building Description	Private University Gymnasium
No. of storeys / basements	1 with mezzanine at end and sides of building
Structural system	Long span steel truss roof supported on concrete frame.
Earthquake resisting system	Transverse: Portal frame (steel truss and concrete columns). Longitudinal: Concrete moment frame
Foundation system	Unknown
Other notable features	Long span steel roof truss is cast into the concrete frame at the roof level beam column joint. End walls are concrete infill between the concrete frame members. Concrete chimney extending approx. 3m above roof is located approximately 1m from side of building
Past seismic strengthening	None
Placard and date (if known)	Red (stickered 1 week following earthquake approx. 27 April 2016)

Photo 1: Aerial view of campus indicating location of Gymnasium.

Photo 2: Building front exterior elevation (note damage to infill concrete walls and ceiling at roof interface).

Photo 3: Building interior (note daylight at end wall roof interface).

Photo 4: Shear cracking damage to concrete beam supporting mezzanine on side wall.

Photo 4: Shear cracking damage to concrete beam supporting mezzanine on side wall.

Photo 5: Light fitting partially unclipped.

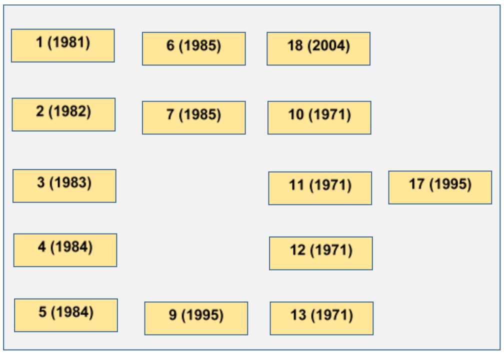
Photos 6 & 7: Chimney adjacent to building (note pounding damage at roof interface).

Observations

- Loss of connection between the front end wall and the roof portal frame observed extending across at least the middle half of the end wall. The steel embedded connections had pulled out of the concrete. Resultant damage to exterior roof ceiling observed.
- Pounding of chimney and roof indicates movement of approximately 1.2m at roof level between the gymnasium roof and chimney.
- Shear damage to beams supporting mezzanine on side walls appeared to be limited to one beam location.
- Connection between roof trusses and concrete frame not able to be observed.
- No damage observed to hung lighting or other hung elements including basketball hoops supports except one light fitting partially unclipped.
- Building used as a shelter for approximately a week before damage assessment completed and damage identified.

Building Damage Report – A8

Item	Details
Street Address/Location	Kumamoto City, Kumamoto Prefecture, South East of Kumamoto Castle
Date of construction	Constructed between 1971 and 2004
Building Description	13 x 5 storey Apartments Buildings
Building Footprint / Floor Area	Each building ~250 m^2
No. of storeys / basements	5 storeys
Structural system	RC moment frame
Earthquake resisting system	RC moment frame with transverse shear walls. Concrete “non structural” infill exterior walls
Foundation system	38m deep, hollow 500 dia. piles 350mm dia. Cavity in pile used to auger through earth and rest of pile drive down simultaneously. Bottom 500mm of pile socketed into dense material and backfilled with grout for increased bearing area at base
Past seismic strengthening	Buildings 10, 11, 12 and 13 have been seismically retrofitted
Likely Design Standards	Buildings 1-9, 17, 18 post 1981 code. Buildings 10-13 pre 1981 code.
Placard and date (if known)	Buildings 3,4 and 5 evacuated all others have green placards



Apartment complex plan view with building numbers and year constructed

Typical building elevation.

Photo 1: Cracking of non-structural walls due to differential settlement of building.

Photo 2: Visible leaning of Building No. 5 (One side noticeably lower than other).

Photo 3: Separation of columns and non-structural walls as part of seismic retrofitting.

Photo 4: Seismic retrofitting of pre-1980 buildings.

Observations

- Buildings 3, 4 and 5 were damaged and evacuated due to global building tilts of between 3 and 4%. These buildings are planned to be demolished and reconstructed.
- Damaged buildings had extensive disruption of soil immediately surrounding each building.
- Some limited cracking of non-structural walls due to differential movement observed.
- Buildings with seismic retrofitting performed well with no visible damage.
- Buildings 3, 4 and 5 are to be demolished and reconstructed.

Building Damage Report – A9

Item	Details
Street Address/Location	Miyazono 702, Mashikimachi, Kamimashiki District, Kumamoto Prefecture
Building Description	Municipal Office Building, connected by small footbridge to adjacent building
No. of storeys / basements	3 Storeys, no basement on sloping site
Structural system	RC moment frame
Earthquake resisting system	RC moment frame
Foundation system	Piled Foundations
Past seismic strengthening	Retrofitted with extra concrete frame on outside of building
Likely Design Standards	Pre 1980 Japanese Design Code
Placard and date (if known)	Green (initially Red)

Building elevation.
Additional exterior RC moment frame added as a seismic retrofit shown.

Photos 1 & 2: Ground subsidence around the building.

Photo 3: Damage to bridge between two buildings.

Photo 4: Damage to bridge between two buildings

Observations

- Significant localised ground deformation observed around the building.
- Bridge Structure between two buildings shows signs of shear cracking in walls and also failure at joint between bridge and columns (refer Photos 3 & 4).
- Building retrofitted with additional concrete frame constructed on building exterior supported on separate piled foundations (refer building elevation).
- Post earthquake cantilevered roof and elevator on side of building have been removed and a new panel cladding has been installed.

Building Damage Report – A10

Item	Details
Street Address/Location	1 Chome 32-35, Obiyama, Chuo district, Kumamoto City
Date of Construction	1965
Building Description	School classroom building
Building Footprint / Floor Area	L shaped building one classroom plus corridor wide. Classrooms on playground side. Adjacent classroom block separated from this building with a seismic gap.
No. of storeys / basements	3 storey , no basement
Structural system	Concrete moment frame with infill concrete “non-structural” walls. Cast in place concrete floors constructed with shallow arch configuration.
Earthquake resisting system	Longitudinal: Concrete moment frame on exterior of building Transverse: Concrete moment frame and concrete infill “non-structural walls between classrooms
Foundation system	unknown
Past seismic strengthening	Steel brace retrofit on longitudinal direction on inside of exterior elevations. More extensive bracing on classroom/playground side compared with corridor/street side of building
Likely Design Standards	Pre 1981 code
Placard and date (if known)	Assessed post earthquake as 95% capacity overall, one column assessed as class IV, other columns class 0 – I. School continuing to operate post earthquake

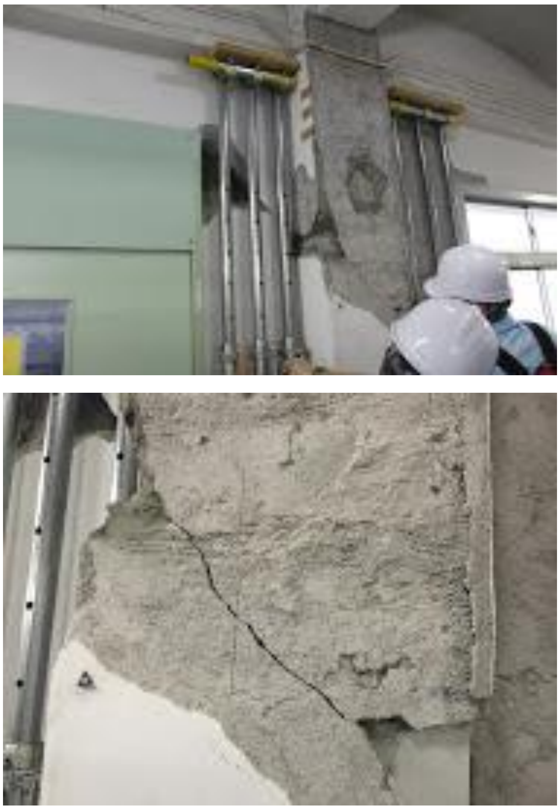
Photo 1: School entrance from street.

Photo 2: School elevation from playground side.

Photo 3: School elevation from playground side.

Photo 4: Interior of corridor showing retrofitted braces.

Photo 5: View of interior corridor. Interior wall of classrooms framed in non-structural aluminium joinery partitions.



Photos 6 & 7: Shear damaged column (damage class IV) located in corridor 1st floor level.

Photo 8: Typical damage to concrete "non-structural" walls between concrete moment frames.

Photo 9: Pounding damage at seismic joint between classroom blocks.

Observations

- Damage relatively limited. Significant shear crack in one column mid-height on corridor level 1, identified in Japanese damage assessment system as class IV. All other columns identified in Japanese damage assessment system as classes 0 or I, no damage or slight cracking.
- Damage observed at seismic joint with adjacent school block.
- Damage observed (cracking of concrete) in the non-structural concrete walls on the building exterior.
- School operating not withstanding shear crack damaged reinforced concrete column and various cracks in "non-structural" concrete walls.

Building Damage Report – A11

Item	Details
Street Address/Location	Kuhonji 3-1-1, Chuo District, Kumamoto City
Date of construction	1960
Building Description	Private High School Classroom building
Building Footprint / Floor Area	2224 m2
No. of storeys / basements	4 storeys, no basement
Structural system	Rectangular regular RC building supported on concrete frame and walls.
Earthquake resisting system	Transverse (2 unequal bays): RC frame with walls between classrooms Longitudinal (9 almost equal bays): RC moment frame
Foundation system	Unknown
Other notable features	The two longitudinal frames had full-width windows, which resulted in short columns. The window height was smaller in the corridor side, which made this frame more critical/vulnerable to short column shear failure. In the transverse direction, RC walls within the larger bay (excluding the corridor) of every alternate frame separated the classrooms. Walls may have been designed to be non-structural, but due to their monolithic construction with the columns they will contribute to the lateral strength and stiffness.
Past seismic strengthening	None
Placard and date (if known)	Red (1 week following earthquake approx. 27 April 2016)
Other	Seismic assessment was conducted before earthquake and the rating (denoted as *Is* as per Japanese evaluation method) was 0.19. Concrete strength was 9.2 MPa

Photo 1: Damaged columns from outside the building

Photo 2: Close up view of typical columns failed in short column induced shear failure.

Photo 3: A typical classroom (Internal longitudinal frame can be seen at the left end of the room, transverse frame without wall can be seen in the middle of the room and transverse frame infilled with structural wall can be seen at the far end of the classroom.

Photo 4: A rare instance of damage to the classroom side external columns.

Observations

- Severe shear damage to the external columns on the corridor side. The cracks were typical of short column failure; i.e. x shaped cracks across the height of the window.
- No similar damage observed on the other side columns as the windows on the opposite side were taller. Also, the transverse shear walls were connected to the alternate columns in the other side, which further restrained the deformation (and hence damage) of the columns on the other side.
- Signs of plastic hinges about to develop at the ends of the beam in the extreme longitudinal frame in some of the classrooms.
- No major damage observed in the transverse frame/wall systems.

Building Damage Report – A12

Item	Details
Street Address/Location	Nagaminehigashi 8-13-1, Higashi District, Kumamoto City
Date of construction	1990s
Building Description	Private Apartment
Building Footprint / Floor Area	Rectangular (roughly: 12mx30m), ~350—400 m2
No. of storeys / basements	12 storey with pilotis ground floor for parking
Structural system	Rectangular regular RC building supported on concrete frames and walls.
Earthquake resisting system	Transverse NS (1 long bay ~12m): 5 RC moment frames. The frame on west and east ends include wall sections of different lengths while the three interior frames have longer spans with relatively small beams suggesting these columns may respond like cantilever columns. Longitudinal EW (4 equal bays): 2 RC moment frames. The south frame is bare on the ground floor whereas the frame on the North side has infill RC walls in 3 bays
Foundation system	Unknown
Other notable features	The column size was 950x950mm at the pilotis floor. We understand the column size reduces from the 2nd floor onwards. May be due to the stringent design requirement to avoid soft story failure in the current Japanese code (design seismic coefficient of 0.55 for soft storey failure). Beams along the longitudinal direction are deeper and wider than the much longer beam along the transverse direction. Two intermediate beams run along the longitudinal direction on top of the ground floor, but no columns or walls were present along these beams.
Past seismic strengthening	None
Likely Design Standards	Latest Japanese Design Code (post 1981)
Placard and date (if known)	Red, April 27.

Photo 1: Building south-east elevation

Photo 2: Building north elevation.

Photo 3: Damage to the columns on South side (most severe damage was to the far-east end column).

Photo 4: Damage to the central beam-column connection on the south frame.

Photo 5: Damaged walls and column on the north side.

Photos 6 & 7: Close up view of typical columns failed in short column induced shear failure.

Photo 8: Beam-Column connection at the top of the shear damaged column.

Photo 9: Shear cracks on the east side wall and damage to the south-east corner column.

Photo 10: Damage to the SE Corner column and its connection to the adjacent East face wall (Photo taken from outside the building).

Photo 11: Damaged orthogonal walls at the NE corner (Photo taken from outside the building).

Photo 12: Close up view of the beam column joint at the NE corner from onside.

Photo 13: NW corner column and adjacent walls.

Photo 14: Column adjacent to the north entrance.

Observations

- The building was observed to have tilted in clockwise direction. Looking from the north side, the building had tilted to about 100mm at the top towards east. (Refer Photo 2 The number of railings crossing the edge of adjacent (left side) undamaged apartment confirmed the slight eastward lean of the building on this side.)
- The walls (we were told they were non-structural, but the Tokyo University researchers were saying these were designed as structural walls) on the North face and the east face were severely damaged.
- North walls had failed in shear, it appeared that the walls were subjected to huge shear towards east.
- The east walls had bidirectional shear cracks throughout the wall
- A column on the north longitudinal frame had failed in shear. Although the bars appeared buckled at the damaged location, it was not the typical flexure-induced buckling of bars. The bars were subjected to shear deformation (dowel action) and axial compression which forced them to buckle.
- There was sign of plastic hinges about to develop in some beams.

Building Damage Report – A13

Item	Details
Street Address/Location	Kumamoto Prefecture
Date of construction	1971
Description / Building Occupancy	University Main Campus Building with a "Y" shaped plan area.
No. of storeys / basements	4 stories
Structural system	RC frame
Earthquake resisting system	RC frame
Foundation system	Piled Foundations
Past seismic strengthening	Building Retrofitted in 2012
Likely Design Standards	Pre 1981 Japanese Design Code
Placard and date (if known)	Placard was not observed but building is heavily damaged and evacuated.

Photo 1: Aerial view of campus indicating location of building.

Photo 2: Building elevation.

Photos 3 & 4: Ground subsidence around building.

Photo 5: Ground subsidence around building.

Photo 6: Shear failure of column at centre of building due to short column effect.



Photos 7, 8 & 9: Visible signs of damage at "Y" juncture of building.

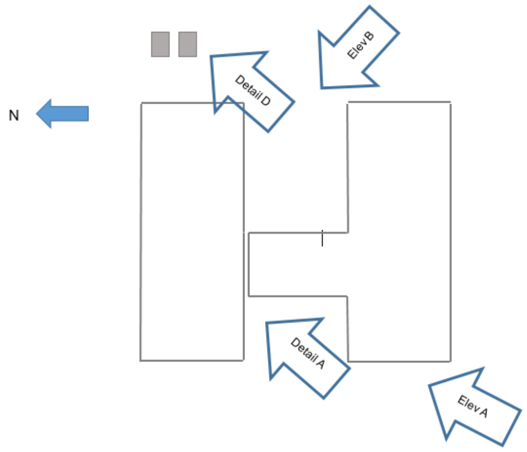

Photos 10 & 11: Cracks found throughout ground on both sides of the building.

Observations

- Building identified as not meeting seismic requirements and retrofitted in 2012 with steel bracing.
- Building damaged at centre where two sides of "Y" shape have been pulled apart (see Photo 4-7).
- Campus located in vicinity of volcanic mountains.
- University campus is located just north of a known fault location. Ground cracks could be indication of the location of the fault or potentially due to ground settlement and slump of sedimentary layer (from previous eruptions) down nearby slopes.
- Damaged columns (refer Photo 4) shows lack of stirrups/ties around internal longitudinal reinforcement (only single stirrup around all longitudinal reinforcing bars).

Building Damage Report – A14

Item	Details
Street Address/Location	Kumamoto Prefecture
Date of construction	Part 1998 and part 2010
Description / Building Occupancy	University teaching, research, office building
Building Footprint / Floor Area	An H shaped building complex comprising two 4 storey wings connected by a two storey interconnector block. Seismic joints separate the tow 4 storey wings at the interconnector.
No. of storeys / basements	4 storeys plus basement
Structural system	Cast in place concrete moment frame building
Earthquake resisting system	Concrete moment frame building with concrete "non-structural" walls
Foundation system	unknown
Past seismic strengthening	None
Placard and date (if known)	Green



Sketch plan of building layout

Elevation A: View of west exterior elevation looking from one 4 storey block towards the other 4 storey block

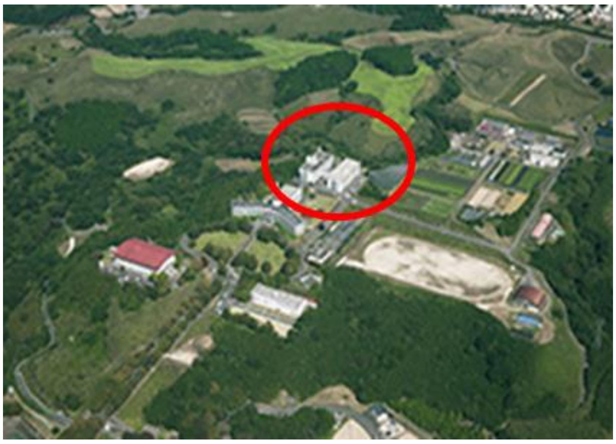


Photo 1: Aerial view of campus indicating location of building.

Elevation B: View of east exterior elevation of one of the 4 storey building blocks and interconnecting block.

Detail A: Seismic joint between four storey block and interconnector at building exterior.

Detail B: Seismic joint between four storey block and interconnector at building interior showing some damage to floor at joint.

Detail C: Interior view of south four storey block at ground level. Note no damage to interiors including ceilings.

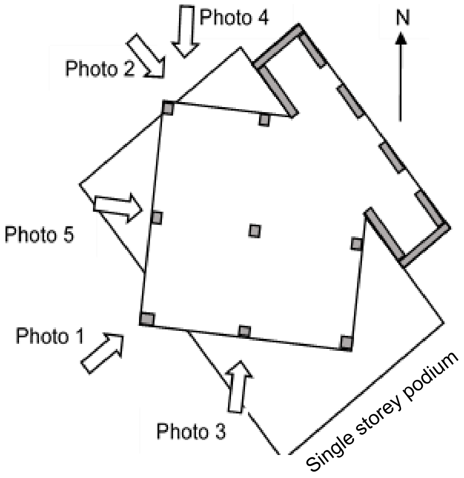
Detail D: Chiller plant located exterior to building. Note ground rupture adjacent to foundations.

Observations

- Minor damage observed at seismic joint between the buildings.
- No damage observed to non-structural elements within the building including ceilings, and MEP
- Advised of shear failure of foundation beams at the seismic joint and one corner of the building advised to have risen by 300 mm.
- Building apparently 0.7% inclined

Building Damage Report – A15

Item	Details
Street Address/Location	Uratamichi 51, Uto City
Age	Unknown. Possibly late 1950s/early 1960s
Description / Building	Municipal Office Buildings
Building Footprint / Floor Area	24x24m main floor area + 24x3m services/access core
No. of storeys / basements	5
Structural system	RC moment-frames around main office floor area. Unknown floor system, possibly two-way flat slab. Service core is perforated RC walls.
Earthquake resisting system	RC MRFs + core walls
Foundation system	Unknown
Other notable features	Core of building is eccentric to main floor area and is at 45 degrees to the grid setout of the main office floor. This results in significant eccentricity and limited slab connection to the core
Past seismic strengthening	None
Likely Design Standards	Pre 1981 code



Plan showing floor slab outline, full-height structural elements and photo locations

Photo 1: South-west elevation.

Photo 2: North-west elevation.

Photo 3: Column plastic-hinge at underside of roof beam and shear failure at Level 4 beam-column joint.

Photo 4: Beam plastic-hinges and permanent offset of upper floors.

Photo 5: Beam-column joint shear failure and dislocation of beam from column.

Observations

- This building suffered severe damage at the fourth floor, while relatively little damage was observed in the lower levels. The building has a very high plan eccentricity due to the services and stair core being located outside the main floor footprint, and attached at one corner of the floor plan.
- It is likely that the beam-column joint shear failure occurred due to accentuated deformation demands under the torsional plan response. Perimeter beam offset from the column centre-line and possibly poorly anchored reinforcement, either in the beam-column joint or from.
- An internal beam, may have allowed the perimeter beam on the south-west elevation to drop inwards from the column, resulting in the 1.5m floor settlement and outward lean of the upper floors to the south-west.
- This building was listed as a high priority for seismic retrofit prior to the earthquakes

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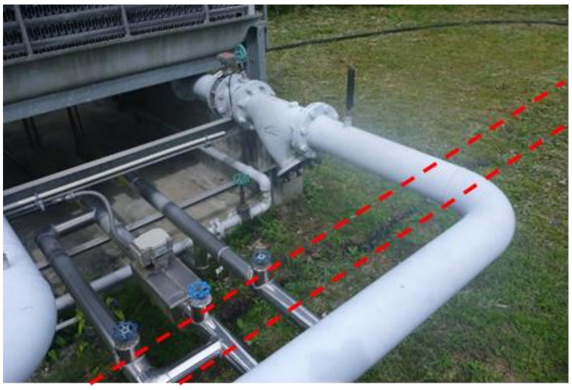
Building Damage Report – A16

The observations below relate to the performance of different non-structural systems and components in the various buildings visited. Refer to the building damage assessments referenced for further details of the buildings.

Building Mechanical Plant



Photos 1 & 2: Building A14 major plant items located behind the teaching / office building.

- Ground rupture observed beside chillers (highlighted in above photos with dashed lines).
- No damage observed to plant or plant supports or connecting piping.
- The arrangement of major plant on the ground near the building was observed at many locations.

Photo 3: Building A5. Plant and piping in the basement located below the plane of isolation and flexible connectors across the plane of isolation.

Photo 4: Building A5. Plant and piping in the basement located below the plane of isolation and flexible connectors across the plane of isolation. Note base isolation dampers in background.

Photo 5: Building A5. Supports for storm water piping located immediately hung from ground floor above the plane of seismic isolation.

- Major services plant located in basement below plane of isolation. Flexible pipe connections at all locations where plant crosses the plane of isolation with braced supports on each side of the flexible connection.
- Storm water plumbing appeared to be located entirely above the plane of isolation on hung supports.
- No damage observed to any of the plant or piping.

Ceilings

Photo 6: Building A7 designed with no ceiling thus avoiding damage

Photo 7: Building A7 detail of ceiling.

- Damage to exterior soffit where end wall of the building separated from the roof structure. No ceiling on building interior.
- Detail of lights hung from roof structure indicating one light fitting that has become partially unclipped, assumed to be resulting from earthquake movements.

Photo 8: Building A13 interior ceiling damage.

- Localised damage to hung ceiling tiles adjacent to structural column penetrating ceiling.

Photo 9: Building A14 building internal ceiling, no damage observed

- No damage observed to ceilings or hung services.
- No fire sprinklers observed.

Photo 10: Building A10 classroom corridor.

- No hung ceiling.
- Acoustical protection applied directly to underside of concrete floor structure. No damage observed to hung services.

Photos 11 & 12: Gym close to Building A10.

- Suspended ceiling damaged at changes in orientation and adjacent to structural end walls and columns. No damage observed to hung services e.g. lights.
- No fire sprinklers observed.
- Note damage at change in ceiling orientation

Photo 13: Building A11.

- No damaged observed to ceiling or hung non-structural elements. Ceiling appeared to be rigidly adhered to the concrete floor structure above.
- No fire sprinklers observed.

Underground Services

Photos 14 & 15: Ground and underground services disruption Building A13.

Photo 16: Broken piping adjacent to Building A13.

- Significant ground rupture observed in the area of the dormitory resulting in significant damage to underground services particularly at junctions between pipes and cast in place concrete chambers and concrete foundation pads.

Building Cladding Systems

Photo 17: Building A14. Exterior covered in ceramic tiles applied directly to the concrete structure. A typical Japanese cladding system

- No damage observed to building cladding system.

Photo 18: Building A12, Exterior tiled cladding.

- Cracking in cast in place concrete wall elements (considered to not be part of the lateral load resisting structure evidenced in the exterior tiles.

Photo 19: Apartment buildings Kumamoto. Undamaged buildings.

- Note separation joint in concrete balcony balustrades to allow movement without damage to building.

Stairs

Photo 20: Building A10 no damage observed to stairs.

- Stairs constructed in reinforced concrete, no sliding joints observed.

Photo 21: Building A11.

- No damage observed to stairs, minor cracking to non-structural wall below stairs.
- Stairs constructed in reinforced concrete, no sliding joints observed.

Furniture / Signage

Photo 22: Building A10.

- Bookcase laid down in school corridor to prevent possible falling in any aftershocks.

Photo 23: Building A13 exterior signage.

- Limited mortar joint affixing stone signage to wall.

Temporary Buildings

Photo 24: Building A11, Temporary school administration block.

Photo 25: Building A15, City Hall temporary office block erected on carpark adjacent to the damaged buildings they are temporarily replacing

Photo 38: Building A11,, Temporary class room block erected in school playing field