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Testing and simulation of 3D effects on progressive collapse resistance of RC buildings

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This paper evaluates the three-dimensional (3D) or slab effects on reinforced concrete (RC) buildings to mitigate progressive collapse, which is caused by the loss of an interior column. Six one-quarter scaled beam-column, or beam-column-slab substructures are tested. These six specimens are categorised into three series (P-, T- and S-series). The test results confirm that transverse beams and RC slabs can reduce the collapse vulnerability of RC buildings effectively. In addition, it is quantified that 3D effects without slab can increase the yield load of the frame by up to 100%, while 3D effects including slab can increase the yield load up by 246.2%. This is because the slabs not only increase the bending moment capacity of beam sections working as flanges, but also provide more alternative load paths for load redistribution. RC slab can upgrade the first peak load of the buildings by developing compressive membrane actions, and upgrade the ultimate load capacity of the building during the large deformation stage by developing a tensile membrane action. As the number of tested specimens is relatively small, a series of numerical and parametric studies are carried out to further quantify the 3D or slab effects on RC buildings in resisting progressive collapse.

Notation

F_{cr}^*	first cracking load
F_t^*	ultimate load capacity
F_u^*	first peak load
F_y^*	yield load
f_c'	compressive strength of concrete
f_t	tensile strength of concrete
f_u	ultimate strength
f_y	yield strength
G_c	compressive fracture energy
S_{DS}	design spectral response acceleration parameters at short periods
S_{D1}	design spectral response acceleration parameters at 1 s period
s	slip
s_1	slip at which bond strength is achieved
s_2	slip at which bond strength begins to decrease
s_3	slip at which mechanical bond resistance is lost
U_x, U_y, U_z	displacement in x, y and z directions, respectively
ε_y	yield strain
$\theta_x, \theta_y, \theta_z$	rotation in x, y and z directions, respectively
τ	bond stress
τ_f	residual bond stress
τ_{max}	maximum bond stress

Introduction

Progressive collapse, characterised by widespread propagation of initial failures at a disproportionately large scale, can be triggered by the loss of a few vertical supporting elements. Although the possibility of progressive collapse events is low, their catastrophic consequences have captured considerable attention from government agencies, research communities and practical designers. To understand the behaviour of structures in resisting progressive collapse, a series of numerical and experimental studies (Astanek-Asl *et al.*, 2001; Choi and Kim, 2011; Fu, 2010; Kaewkulchai and Williamson, 2004; Kim and Yu, 2012; Marjanishvili and Agnew, 2006; Qian and Li, 2012a, 2013a, 2013b; Qian *et al.*, 2014; Sadek *et al.*, 2011; Sasani and Sagioglu, 2008; Su *et al.*, 2009; Yap and Li, 2011; Yu and Tan, 2014) has been conducted by national defence agencies and academic researchers. Previous studies have evaluated the reliability of seismic design for upgrading reinforced concrete (RC) structures to resist progressive collapse and the reliability of compressive arch action (CAA) or tensile catenary action (TCA) developed in RC beams to provide additional resistance capacity for collapse mitigation. However, the majority of previous studies have related to two-dimensional (2D) planar frames or sub-frames and have not incorporated the three-dimensional (3D) or slab effects. Li and El-Tawil (2011) studied the 3D effects on composite structures

under loss of column scenarios by way of numerical simulation. It was found that 3D effects play a very important role in determining the response of the structure subjected to the sudden column loss scenario. The study confirmed that transverse beams and RC slab contribute considerable additional strength and stiffness. Thus, they reduce the progressive collapse vulnerability of buildings significantly. However, it was indicated that the slab may enlarge the collapse zone if collapse could not in the end be prevented. This was mainly owing to tensile membrane action (TMA) developed in the RC slab, which pulled the surrounding elements to collapse as a result of substantial in-plane tensile force. Qian and Li (2012b) studied slab effects on the performance of RC substructures in mitigating progressive collapse experimentally. It was found that slabs could upgrade the first peak load resistance capacity up by 63%. However, only the scenario where a corner column was missing was investigated by Qian and Li (2012b). Thus, the contribution of TMA and compressive membrane action (CMA) developed in RC slabs to resist collapse could not be thoroughly investigated owing to the limited horizontal constraints provided by surrounding components for RC frames under the loss of a corner column scenario. Therefore, in order to carry out systematic assessment of the reliability of CAA and TCA developed in RC beams and CMA and TMA developed in RC slabs for preventing collapse of RC buildings, three series of specimens were designed and tested; these are described by Qian *et al.* (2014) in detail. However, the extent of 3D effects is different for buildings designed with varying dimensions and reinforcement properties. Thus, a series of numerical and parametric analyses, which were validated by test results from Qian *et al.* (2014), were carried out to further quantify the 3D or slab effects on progressive collapse resistance of RC buildings.

Observed experimental results

The experimental programme was presented in Qian *et al.* (2014) in detail, but for the sake of easier understanding, the test set-up, specimen design and test results are introduced briefly herein. Two prototype structures were designed in accordance with ACI

Committee 318-08 (2008), with difference span aspect ratios. The design dead and live loads are 6.2 and 3.0 kN/m², respectively. The design spectral response acceleration parameters at short periods, S_{DS} , and at 1 s period, S_{D1} , are 0.45g and 0.34g. A design compressive strength of concrete of 25 MPa was utilised for prototype structures. The yield strength of reinforcements in the prototype structures was 460 MPa. In view of the limitations relating to laboratory spacing and capacity of test facilities, one-quarter scaled models of the part enclosed in the dotted curve in Figure 1 were cast and tested. Dimensions and reinforcement details of the test specimens are shown in Table 1. The beam and slab reinforcement placements of specimen S1 are depicted in Figures 2 and 3, respectively. Specimens S1 and S2 are directly scaled down from prototype buildings, while specimens T1 and T2 respectively have similar beams to S1 and S2 but without RC slab. As shown in Figure 2, the edge beams connected with corner columns had a limited influence because no slab is incorporated and fixed boundary conditions are applied to the adjacent columns. Therefore, no edge beams and corner columns are cast in specimens T1 and T2. Specimens P1 and P2 are planar frames and can be seen as the longitudinal and transverse beams

Figure 1. Plan layout of a prototype building

Test ID	Elements		Longitudinal rebar			Transverse reinforcement ratio			Slab reinforcement ratio	
	Beam-T	Beam-L	Column-I	Beam-T	Beam-L	Joint	Beam-T	Beam-L	Top	Bottom
P1	Type 1*	NA	4T13	4T10	NA	R6@60	R6@140	NA	NA	NA
P2	NA	Type 2*	4T13	NA	4T10	R6@60	NA	R6@140	NA	NA
T1	Type 1*	Type 2*	4T13	4T10	4T10	R6@60	R6@140	R6@140	NA	NA
T2	Type 1*	Type 1*	4T13	4T10	4T10	R6@60	R6@140	R6@140	NA	NA
S1	Type 1*	Type 2*	4T13	4T10	4T10	R6@60	R6@140	R6@140	0.25%	0.25%
S2	Type 1*	Type 1*	4T13	4T10	4T10	R6@60	R6@140	R6@140	0.25%	0.25%

Note: Type 1*: clear span = 1300 mm, cross-section = 140 × 80 mm²; Type 2*: clear span = 1900 mm, cross-section = 180 × 100 mm². T10 = deformed bar of 10 mm diameter; Beam-L = longitudinal beam; Beam-T = transverse beam; Column-I = interior column.

Table 1. Specimen properties (unit: mm)

Figure 2. Dimensions and reinforcement details of S1

of specimen T1, respectively. The measured average concrete compressive strengths at 28 d after casting are 20, 21, 22, 23, 23 and 23 MPa for P1, P2, T1, T2, S1 and S2, respectively. As shown in Figures 2 and 3, T10, T13 and T16 were utilised for the

main reinforcing steel of beams and columns while R6 steel bars were utilised for stirrups and slab rebar. T and R represent deformed and plain rebar, respectively. The measured properties of these steel bars are shown in Table 2.

Figure 3. Slab reinforcement details of S1

Types	Yield strength, f_y : MPa	Yield strain, ϵ_y : $\times 10^{-6}$	Ultimate strength, f_u : MPa	Ratio of elongation: %
T10	437	2273	568	13.1
T13	535	2605	611	11.6
T16	529	2663	608	14.3
R6	355	1910	465	17.5

Note: R6 = plain round bar of 6 mm diameter; T10 = deformed bar of 10 mm diameter. T13 = deformed bar of 13 mm diameter; T16 = deformed bar of 16 mm diameter.

Table 2. Properties of reinforcing steel

As shown in Figure 4, for specimens S1 and S2, eight strong steel supports are designed to provide fixed boundary conditions to surrounding columns. No support is provided beneath the centre column to simulate the loss of centre column caused by extreme events (whether accidental or intentional). A hydraulic jack is used to apply vertical displacement at the centre column to achieve a push-down loading regime. A steel assembly beneath the hydraulic jack is specially designed to ensure that the applied

load is vertical and the failure of the specimens is symmetric. For P- and T-series specimens, only two and four supports are respectively utilised to apply fixed conditions to adjacent columns. Series of linear variable deformation transducers (LVDTs) and line transducers are installed before tests to monitor the varying deformation. Strain gauges are also mounted on the steel bars before casting. The locations of strain gauges on the slab steel bars are shown in Figure 3. For a detailed description of the test set-up, please refer to Qian *et al.* (2014).

Figure 5 shows the deformed shape and failure pattern of specimen S1, which has the span aspect ratio of 1.4. At the end of the test, severe flexural cracks with elliptical shape can be observed at the top face of the slab and concrete crushing is observed at the top slab near the centre column. The majority of the deformation is concentrated in the centre of the slab (enclosed by the outermost elliptical crack), while limited deformation and damage are observed in the outer region. The inner region with large deformation is termed tensile net, whereas the outer region is called compressive ring. From the bottom view, it can be observed that severe cracks occurred at beams as well as slabs. Steel rebar is fractured at the centre column–slab interfaces. Punching shear failure is observed in the center column–slab connection and severe detachment between slabs and beams is also observed. Similar observations are obtained for S2 except that circular cracks are formed at the top face and concrete crushing is observed at the compressive ring at the end of the test, as shown in Figure 6. For specimen T1, as shown in Figure 7, plastic hinges are formed at the beam ends, accompanied by flexural cracks and concrete crushing. Rebar fracture also occurs at the interior column–beam interfaces. Unexpectedly, severe shear damage is observed at one of the transverse beams. This may be due to initial deficiencies concentrated in that region before testing. However, the shear failure did not stop the development of tensile catenary action in the beams. For specimens T2, similar behaviour is observed. Figure 8 presents the failure mode of specimen T2 at the end of the test. For specimens P1 and P2, their failure modes are similar to the longitudinal and transverse beams of T1, respectively. The failure mode of specimen P1 is shown in Figure 9.

Figure 10 shows the load–displacement relationships of test specimens, and Table 3 presents the key results for the test specimens. It can be seen that the first peak loads of tested specimens exceeded their yield loads significantly, mainly owing to CAA and CMA developed in beams or slabs. The ratios of first peak load to yield load are 1.33, 1.38, 1.40, 1.33, 1.44 and 1.37 for specimens P1, P2, T1, T2, S1 and S2, respectively. Moreover, re-ascending of load resistance is observed in all specimens. For the S-series of specimens, the load re-ascending was observed at about 1.2*t*. However, for P- and T-series of specimens, the load re-ascending was observed at about 1.0*d*. It should be noted that *t* and *d* represent slab thickness and beam depth, respectively. The ultimate load

Figure 4. An overview of test specimen in position ready for testing: (a) specimen S1; (b) specimen T1

capacity due to developing TCA and TMA could increase the first peak load by 47%, 64%, 18%, 41%, 47% and 34% for specimens P1, P2, T1, T2, S1 and S2, respectively. To quantify the 3D effects, the test results from T-series tests are compared with P-series tests. It is found that 3D effects excluding slab contribution could increase the first crack load, yield load, initial stiffness, first peak load and ultimate load capacity by up to 88%, 100%, 127%, 109% and 68%, respectively. Comparing the test results of S-series with P-series indicated that 3D effects including slab contribution could increase the first crack load, yield load, initial stiffness, first peak load and ultimate load capacity up by 450%, 246%, 244%, 259% and 260%, respectively.

Although the test results confirmed that ignoring slab effects may result in extremely conservative evaluation, the above values are only suitable for buildings with similar dimensions and reinforcement details as the test specimens. It is understandable that varying the dimensions and reinforcement details may result in the efficiency of 3D and slab effects differing significantly. With the consideration of time and cost limitations, investigating all parameters by experimental tests is not realistic and validated finite-element analyses (FEAs) may be taken as an alternative method.

Finite-element analysis

General

The following sections present a non-linear finite-element numerical study carried out to further quantify the 3D and slab effects for structures with different dimension or reinforcement details.

Element and material models

Concrete

The concrete of beams and columns is modelled using 20-node brick elements (Diana element CHX60), while the slabs are modelled by eight-node shell elements (Diana element CQ40S) (Diana, 2008). The constitutive model used for concrete is the total strain rotating crack model, which is a 3D extension to the modified compression field theory proposed by Selby and Vecchio (1993). According to this model, a crack is assumed to be initiated perpendicular to the major principal stress if its value exceeds the concrete tensile strength, independent of the value assumed by the other principal stresses. The influence of lateral cracking on compression behaviour is taken into consideration by the model proposed by Vecchio and Collins (1986). Moreover, in considering that the total strain rotating crack model had relatively high sensitivity with respect to element size, the maximum dimension of

(a)

(b)

Figure 5. Failure mode of S1 at the end of test

meshed element is 15 mm in this numerical study, based on mesh sensitivity analysis. The response of concrete in compression was modelled by a parabolic compressive softening model. It should be noted that a parabolic compressive softening model is defined by compressive fracture energy G_c and compressive strength f'_c . For details of the model, please refer to Li *et al.* (2009).

In the tension part, conventional linear softening stress–strain based on fracture energy is adopted. As splitting tests had not been carried out, the tensile strength of concrete f_t used in the analysis was determined from the compressive strength f'_c in accordance with CEB–FIP model code 1990 (CEB, 1993)

$$1. \quad f_t = 0.30(f'_c)^{2/3} (\text{MPa})$$

Reinforcement

The Von Mises yield criterion with isotropic strain hardening and an associated flow rule were used to describe the constitutive

(a)

(b)

Figure 6. Failure mode of S2 at the end of test: (a) top view; (b) bottom view

behaviour of steel reinforcements. The strain hardening ratio was assumed to be 0.3 for R6 and T10, but 0.15 for T13 and T16 according to the test results. The failure strains of R6, T10, T13 and T16 are respectively defined as 0.18, 0.13, 0.12 and 0.14. Reinforcement in the slab is modelled by planar grid reinforcement embedded in the shell elements, so-called mother elements. The reinforcement strains of slab reinforcements are computed from the strains of the mother elements with perfect bonding between reinforcement and surrounding elements. The beam longitudinal reinforcing bars are modelled using truss elements (Diana element L6TRU). The bond–slip between the reinforcement element and the surrounding concrete element is modelled by line interface element (Diana element L8IF). The bond–slip model suggested by CEB–FIP model code 1990 (CEB, 1993) was used in this numerical study, as shown in Figure 11 and mathematically shown below

$$2. \quad \tau = \tau_{\max} \left(\frac{s}{s_1} \right)^\alpha \quad \text{for } 0 \leq s \leq s_1$$

$$3. \quad \tau = \tau_{\max} \quad \text{for } s_1 \leq s \leq s_2$$

Figure 7. Failure mode of T1 at the end of test

Figure 8. Failure mode of T2 at the end of test

Figure 9. Failure mode of P1 at the end of test

Figure 10. Comparison of the load-displacement relationship of test specimens: (a) specimen P1; (b) specimen P2; (c) specimen T1; (d) specimen T2; (e) specimen S1; (f) specimen S2

$$4. \quad \tau = \tau_{\max} - (\tau_{\max} - \tau_f) \frac{s - s_1}{s_3 - s_2} \text{ for } s_2 \leq s \leq s_3$$

$$5. \quad \tau = \tau_f \text{ for } s \geq s_3$$

where τ is the bond stress, $\tau_{\max}$ is the maximum bond stress, s is the slip, s_1 is the slip at which bond strength is achieved, s_2 is the slip at which bond strength begins to decrease, and s_3 is the slip at which mechanical bond resistance is lost.

Geometry models

As shown in Figure 12, specimens P1, P2, T1 and T2 are fully modelled, while specimens S1 and S2 are modelled at one-quarter scale to reduce the computational effort required. The dimensions of beams, column and slabs are modelled similarly to the test specimens, except for the simulation of steel support. In order to eliminate divergence occurring in the interface between the steel support (hollow, circular section) and rectangular column stub of test specimens, the hollow, circular steel support was equivalent to a solid rectangular support by keeping EI similar to the hollow steel section.

Test	F_{cr}^* : kN	F_y^* : kN	Initial stiffness: kN/mm	F_u^* : kN	F_t^* : kN	F_u/F_y	F_t/F_u
P1	8	24	1.5	32	47	1.33	1.47
P2	11	26	1.8	36	59	1.38	1.64
T1	15	48	3.4	67	79	1.40	1.18
T2	18	48	3.9	64	90	1.33	1.41
S1	44	80	4.8	115	169	1.44	1.47
S2	56	90	6.2	123	165	1.37	1.34
T1/P1	1.88	2.00	2.27	2.09	1.68	N.A	N.A
T2/P2	1.64	1.85	2.17	1.78	1.53	N.A	N.A
S1/P1	5.50	3.33	3.20	3.59	3.60	N.A	N.A
S2/P2	5.09	3.46	3.44	3.42	2.80	N.A	N.A

Note: F_{cr}^* , F_y^* , F_u^* and F_t^* represent the first cracking load, yield load, the first peak load and ultimate load capacity due to tensile catenary or tensile membrane actions, respectively.

Table 3. Test results

Figure 11. Bond-slip model by CEB-FIP model code 1990 (CEB, 1993)

Solution procedure

The Newton-Raphson method was applied in solving the non-linear equation system. A line search method was applied to increase the convergence speed for the iterations. The convergence criteria of the iterations were based on the energy norm and a maximum of 50 iterations were specified.

Verification of finite-element models

In order to verify that the finite-element models (FEMs) are able to simulate the behaviour of the experimental control specimens properly, two items from the experimental and numerical results were compared. They were load-displacement behaviour and strain development in beam longitudinal reinforcement.

The measured load-displacement curves of test specimens are compared with the simulated curves in Figure 10. As shown in the figure, the FEM could simulate the initial stiffness, yield strength and first peak load very well. For large displacement

Figure 12. Geometric models of finite-element analysis: (a) specimen T2; (b) specimen S2

behaviour (TCA or TMA), the FEM could capture the general trends very well. However, the FEM could not predict the ultimate displacement and failure of the specimens properly. This was mainly because the elements, which had reached their failure criteria were not removed from the model. In this FEA, the authors assumed the specimens had failed when the ultimate displacement had reached double that of the short span.

Figure 13 compares the experimental and numerical strain distributions along the beam longitudinal reinforcement of specimen T2. The distributions were compared at several displacement stages: yield load, first peak load, when catenary or tensile membrane actions had begun to develop, and at two large displacement stages. As shown in the figure, the distributions generally match reasonably accurately both for top and bottom beam longitudinal reinforcements. Thus, it is clear that these models are sufficiently accurate to predict the influence of critical parameters on progressive collapse resistance of RC frames, despite limiting predictions of the final failure stage.

Boundary conditions sensitivity

For simplicity, an equivalent fixed-boundary condition is applied on the adjacent columns or corner columns for the test specimens in this study. To complement the experimental study, the effects of boundary condition simplification on the test results are studied by validated numerical models.

P- and T-series of specimens

For P- and T-series of specimens, the more realistic boundary conditions applied on the adjacent columns should appear as Figure 14(a). Several previous researchers (Qian and Li, 2012c; Yap and Li, 2011) simplified the boundary conditions by applying pin supports on the mid-span of beams and mid-height of adjacent columns, as shown in Figure 14(b). As the test models are three dimensional, such as S-series specimens, equivalent fixed boundary conditions are applied on the adjacent columns in this study, as shown in Figure 14(c). In order to quantify the influence of boundary conditions simplification on the CAA and TCA developing in the beams, validated FEMs with different

Figure 13. Comparison of rebar strain distribution for beam of specimen T2: (a) top reinforcement; (b) bottom reinforcement

Figure 14. Boundary conditions: (a) realistic boundary conditions; (b) pin supports at the middle height of columns and mid-span of beams; (c) equivalent fixed in adjacent column (utilised in this study)

boundary conditions were simulated and their finite-element results are shown in Figure 15. As shown in Figure 15(a), the FEM with equivalent fixed boundary conditions would slightly over-estimate CAA, but could predict the TCA developed in the beams well. By comparing the numerical results of the FEM with pin supports with the results of FEM with realistic boundary conditions, it was found that the pin supports simplification could simulate the CAA accurately, whereas it would underestimate TCA significantly. Figure 15(b) shows the relationships of

horizontal movement plotted against vertical displacement of adjacent joints from FEMs with different boundary conditions. As shown in the figure, similar maximum outward displacements (negative value) and maximum inward displacements (positive value) were obtained from the FEMs with equivalent fixed conditions or realistic boundary conditions. This further proved the reliability of boundary conditions applied on the test specimens. However, the authors suggested designing the stiffness of the supporting elements (equivalent fixed boundary conditions) by validated FEM in future tests.

S-series of specimens

In this experimental study, the rotational constraints of the surrounding slab are conservatively simulated by hanging weights at the edges of the slab (Figure 12(b)). However, in real structures, the surrounding slab not only can provide rotational constraints, but also provides significant horizontal constraints to the test slab. Thus, in order to evaluate this effect, another model with pin-line at the slab edge was simulated by validated FEM. The comparison of the load–displacement curve of slab with free and constrained edges is shown in Figure 16. As shown in the figure, specimen S2 with constrained edges achieved a higher first peak load compared to the slab with free edges. However, a similar large displacement response was observed in the slab with constrained and free edges. In order to further improve the accuracy of the test models, the authors suggest applying vertical and horizontal constraints at the edges of the slab for future tests.

Parametric study

As only six specimens were tested in the experimental programme owing to cost and time considerations, it is impossible to systematically understand the influences of all critical parameters by experimental study. Thus, several critical parameters were studied by the validated FEA. In this parametric study, the influences of each parameter are discussed within two aspects: (a) influences on the first peak load (FPL) and ultimate load capacity (ULC); (b) influences on the extent of slab effects. Specimens T2 and S2 are taken as control specimens. The variables ‘BL’, ‘BD’, ‘SR’ and ‘ST’ represent beam reinforcement ratio, beam depth, slab reinforcement ratio and slab thickness, respectively.

The influences of beam longitudinal reinforcement ratio

Figure 17 illustrates the influence of beam longitudinal reinforcement ratio on the response of bare-frame specimen (T2) and beam–slab specimen (S2). As shown in Figure 17(a), for bare-frame specimen T2, increasing the beam longitudinal reinforcement not only increased the initial stiffness and yield strength, but also increased the FPL and ULC. Similar to the bare-frame specimen, the general behaviour of beam–slab specimen upgrades significantly when the beam longitudinal reinforcement ratio is increased, as shown in Figure 17(b). However, the failure of beam–slab specimen with high longitudinal reinforcement ratio showed a more brittle performance than that with low longitudinal reinforcement ratio. This is mainly due to concrete

Figure 15. Influences of boundary conditions applied on the beam ends: (a) load–displacement curve; (b) horizontal movement plotted against vertical displacement

Figure 16. Influences of boundary constraints applied on the edges of slab

crushing in the compressive zone occurring earlier when the beams had higher longitudinal reinforcement ratio. By comparing these two series of specimens, it was found that the extents of slab effects on FPL and ULC decreased with increase of the beam longitudinal reinforcement ratio, as shown in Figure 18.

The influences of beam depth

As shown in Figure 19(a), increasing the beam depth of T2, the initial stiffness, yield strength will increase. However, the force enhancement factor, defined as the ratio of FPL to yield strength, decreases with increasing the beam depth. Moreover, as shown in the figure, similar ULC was obtained even when the beam depth increased significantly. For beam–slab specimen S2, similar trends were also observed, as shown in Figure 19(b). Comparison of these two series of specimens (T2-BD and S2-BD), as shown in Figure 20,

Figure 17. Influences of beam longitudinal reinforcement ratio on the response of test specimens: (a) specimen T2; (b) specimen S2

Figure 18. Slab effects on RC buildings with varying beam longitudinal reinforcement ratio

indicated that the extent of slab effects on FPL and ULC decreases with increase of the beam longitudinal reinforcement ratio.

The influences of slab reinforcement ratio

Figure 21 illustrates the influences of slab reinforcement ratio on the response of beam–slab specimen S2. As shown in the figure, increasing the slab reinforcement ratio will not affect the CMA substantially but the TMA will increase considerably. Figure 22 indicates that increasing the slab reinforcement ratio will increase the ULC significantly, while no obvious effects are seen on the FPL of the frame.

The influences of slab thickness

Figure 23 shows the effects of slab thickness on the response of beam–slab specimen S2. As shown in the figure, the FPL of S2

Figure 19. Influences of beam depth on the response of RC frames: (a) specimen T2; (b) specimen S2

Figure 20. Slab effects on RC buildings with varying beam depths

Figure 21. Influences of slab reinforcement ratio on the response of bare frames and beam-slab frames

Figure 22. Slab effects on RC buildings with varying slab reinforcement ratio

Figure 23. Influences of slab thickness on the response of bare frames and beam-slab frames

increases with increasing slab thickness, whereas the ULC of S2 decreases with increase of the slab thickness. This is mainly because the thicker slab improved the efficiency of CMA. However, the specimen with thinner slab was prone to deform and to form steel net at the large displacement stage. As predicted, the extent of slab effects on FPL increases significantly with increasing the slab thickness, as shown in Figure 24. However, the extent of slab effects on ULC decreases with increase of the slab thickness.

Figure 24. Slab effects on RC buildings with varying slab thicknesses

Conclusions

Based on the experimental and numerical studies, the following conclusions can be drawn.

- (a) The experimental results indicated that 3D effects excluding slab contribution could increase the yield load of 2D bare-frame by 100.0%. However, 3D effects including slab contribution could increase the yield load of 2D bare-frame by 246.2%.
- (b) The experimental results determined that compressive arch action could increase the yield load of bare-frame by 40%, while tensile catenary action could increase the yield load of bare-frame by 126.9%. For beam–slab frames, the compressive arch action and compressive membrane action could increase their yield load by 44%, while tensile catenary action and tensile membrane action could increase the yield load by 111.3%.
- (c) In general, it was found that the finite-element model could simulate the global behaviour of RC components for progressive collapse well, although the model could not predict the ultimate displacement and the failure of the specimens properly. The study of boundary conditions sensitivity indicated that applying fixed boundary conditions on the beam ends near to the adjacent columns is reasonable, especially for T- and P-series of specimens. However, simulating the effects of surrounding continual slabs on the test slabs by applying weights on the extending part of the slab will result in conservative values, especially for CMA developed in RC slabs.
- (d) Parametric study by validated finite-element models indicated that increasing beam longitudinal reinforcement will significantly improve the overall response of RC structures in resisting progressive collapse. However, beam depth only had a significant effect on the TAA, but no effect on TCA. Moreover, the parametric study also indicated that slab reinforcement ratio had significant effects on TMA but the effects were limited on CMA. Conversely, the slab thickness will affect CMA significantly.

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