Experimental Investigation and Numerical Analysis of New Multi-Ribbed Slab Structure

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ABSTRACT:
Multi-ribbed slab structure (MRSS) is a new type of composite structural system which is composed of prefabricated multi-ribbed composite wall slab and cast-in-place floor slab as well as cast-in-suit concealed outer frame while the multi-ribbed composite wall slab is composed of reinforced concrete frame and built-in infill silicate blocks or light-weight infill panels. This paper presents the results of experimental investigation and numerical analysis results of MRSS. The specimen is four scaled models in which there are three 1/2 scale two-storey two-bay models and one 1/3 scale three-storey two-bay model. The pseudo-static experiments of the specimens have been executed. The nonlinear numerical analysis of four specimens is carried out with two simplified analysis models proposed in the paper. The analyses show that numerical results are good coincidence with the experimental results. The pseudo-static tests and nonlinear numerical modelling show that the multi-ribbed slab structure has a good earthquake-collapse resistant capacity.

Keywords: multi-ribbed slab structure; pseudo-static experiment; nonlinear numerical analysis

1. INTRODUCTION

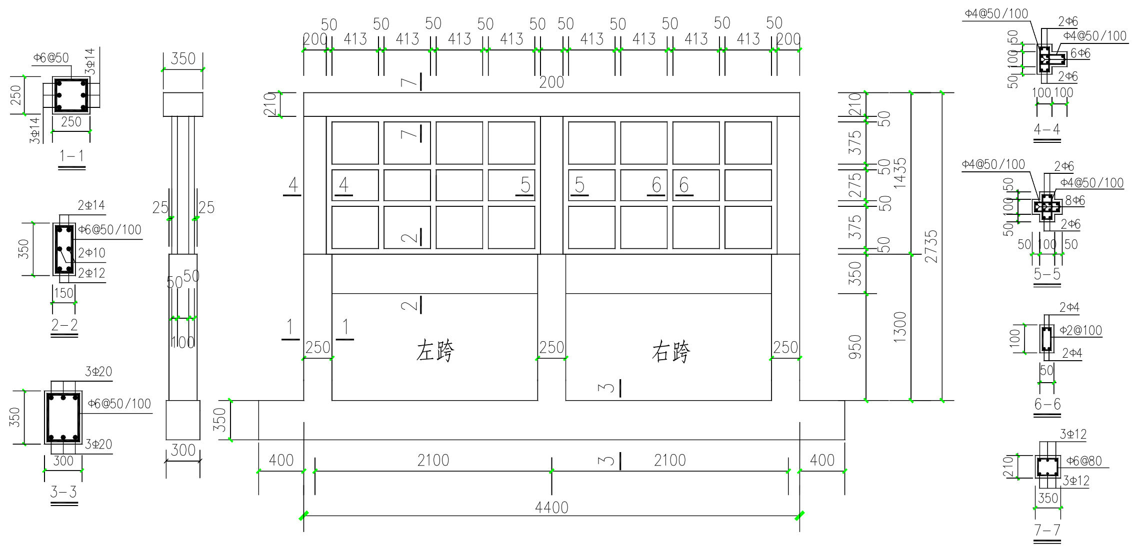
Multi-ribbed slab structure (MRSS) is a new type of composite structural system which is composed of prefabricated multi-ribbed composite wall slab and cast-in-place floor slab as well as cast-in-suit concealed outer frame, in which the multi-ribbed composite wall slab is composed of reinforced concrete frame made up of rib beams and rib columns and built-in infill silicate blocks or light-weight infill panels [1-2]. This paper presents the results of experimental investigation and numerical analysis of MRSS. The specimens are four scaled models in which there are three 1/2 scale two-storey two-bay models and one 1/3 scale three-storey two-bay model. The pseudo-static experiments of the specimens have been executed. The failure phenomena, failure modes, bearing capacity and hysteretic property of the specimens were studied. The nonlinear numerical analysis of four specimens was carried out. Two simplified analysis models such as an equivalent strut model and shear wall model were proposed. The rationality and accuracy of numerical models were validated through comparing with the experimental results. The analyses show that numerical results are good coincidence with the experimental results, especially for forecast of the values of load carrying capacity at feature points. The pseudo-static tests and nonlinear numerical analysis show that the multi-ribbed slab structure has a good earthquake collapse resistant capacity.

2. EXPERIMENTAL INVESTIGATION

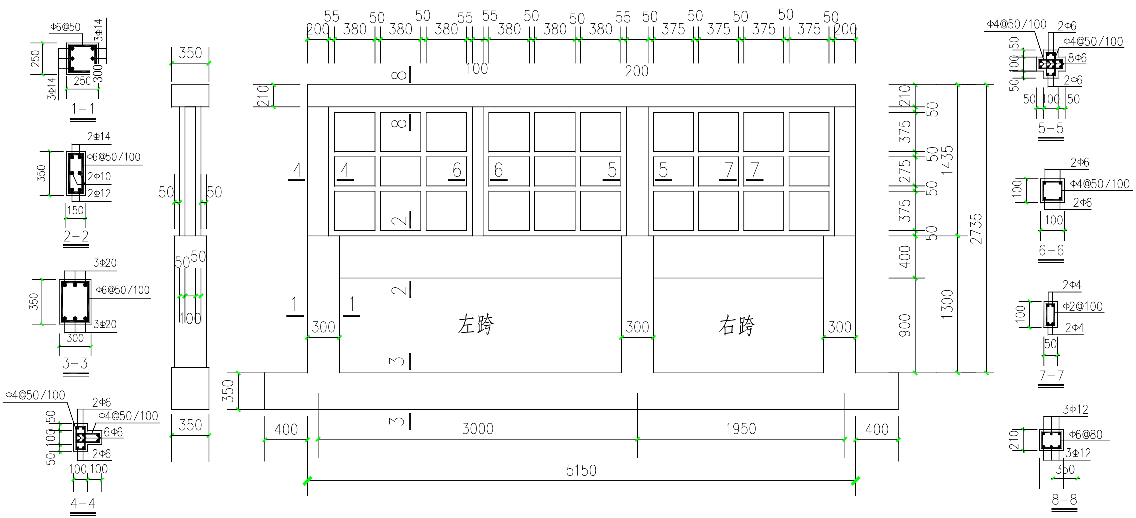
2.1 Specimen Design

The prototype of specimen is an eight-storey frame-supported multi-ribbed slab structure designed according to China building criterion [3-4]. For simplicity, specimens were designed to be a plane structures where bottom frame was one storey and upper multi-ribbed slab structure was 1~2 storey.

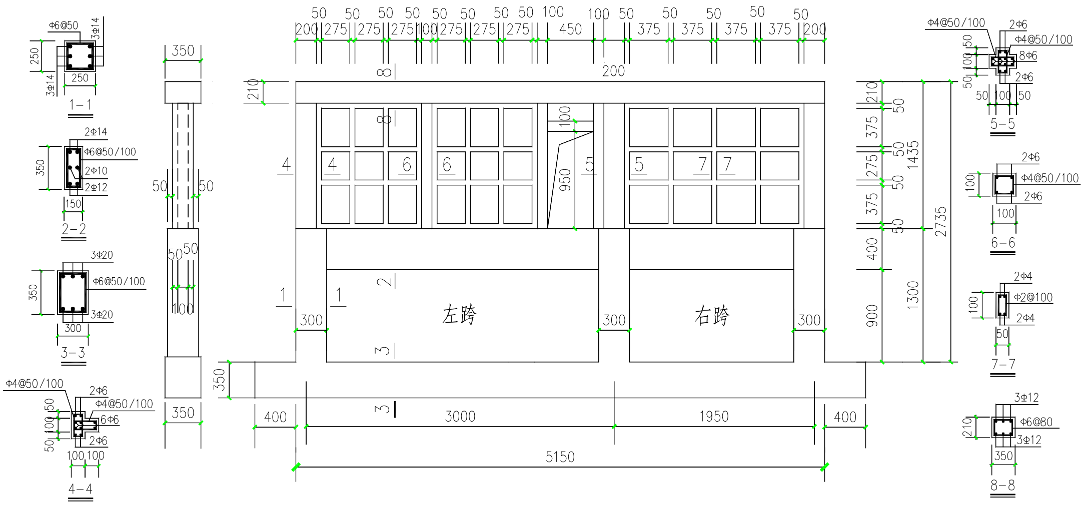
The serial numbers of specimens were KZML-1, KZML-2, KZML-3 and KZML-4 respectively. KZML-1 was a 1/2scale two-storey two-equal-bay models. KZML-2 was a 1/2scale two-storey two-unequal-bay models. KZML-3 was a 1/2 scale two-storey two-unequal-bay models with hollow. KZML-4 was a 1/3 scale three-storey two-unequal-bay models. The wall thickness of 1/2scale models was 100mm while 1/3 scale model was 75mm. The materials of specimen were homologous with prototype. The construction and dimensions as well as reinforcement details of specimens were shown in Figure1.



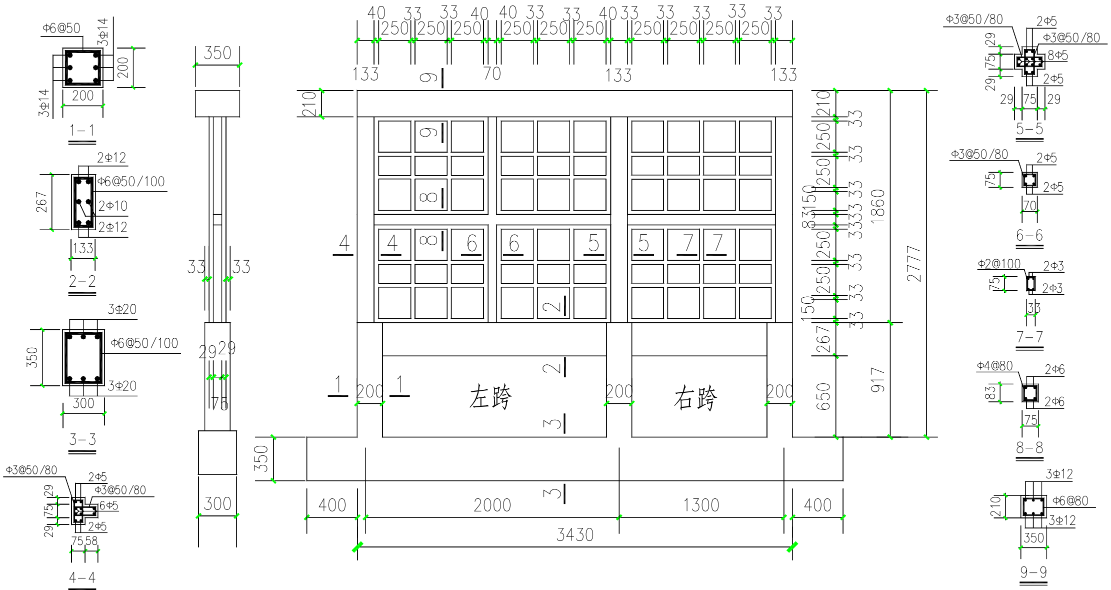
(a) KZML-1



(b) KZML-2



(c) KZML-3



(d) KZML-4

Figure1. The construction and dimensions as well as reinforcement details of specimens

2.2. Loading System

The pseudo-static experiments of the specimens have been carried out. The test loading system was shown in Figure 2. Firstly, vertical loading 331kN, 385kN, 385kN and 236kN were loaded on KZML-1, KZML-2, KZML-3 and KZML-4 respectively. Secondly, horizontal loading were loaded by using loading-displacement mixing control method, in which loading control was adopted by using monotone loading with every step of 10kN/20kN before the specimens yielding, and then after yielding displacement control was adopted with multiples of yielding displacement cycling three times under every step.

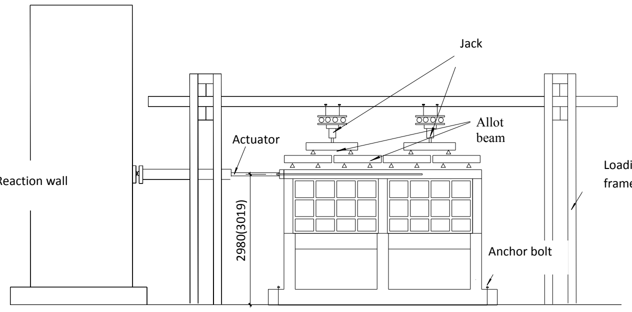


Figure 2. Loading system

2.3. Failure Procedure

Failure phenomena of the specimens are shown in Figure 3.The failure modes of the specimens presented shearing failure characteristic. Under low cyclic loadings, for specimen KZML-1, at initial stages of loading, infilled silicate blocks of multi-ribbed wall slab appeared several slight cracks; loading to metaphase, the cracks of infilled silicate blocks increased markedly, and extended to reinforced concrete rib beams and rib columns; continuing loading, there is a small cracking and slipping between interface of the wall and trimmer beam; loading to yielding of the specimen, interface of the wall and trimmer beam arose big cracking and apparent slipping; failure of specimen KZML-1 mainly took place in the bottom frame post, occurring of post root plasticity hinge brought on serious deformation resulting in specimen not being continued loading. Failure of specimen KZML-2, KZML-3 and KZML-4 mainly concentrated on second storey while damage of frame post was slight; failure procedure of second storey wall took place in the order of infilled silicate blocks, reinforced concrete rib beams and rib columns, outer frame until the infilled silicate blocks desquamated in a big area, the wall degenerated into small frame, end of numerous rib beams came into being plasticity hinges, and local concrete of outer frame post root was crushed partly.

(a)KZML-1

(b) KZML-2

(b)KZML-3

(b) KZML-4

Figure 3. Failure phenomena of specimens

3. NUMERICAL ANALYSIS

3.1. Structural Analysis Model

There are two simplified models that are an equivalent strut model and shear wall model to be proposed for multi-ribbed wall slab.

(a)The equivalent strut model, considering a fundamental infilled frame element with infill silicate blocks in multi-ribbed wall slab shown in Figure 4(a), based on the model of masonry infill panel (Madan and Reinhorn et al., 1997)[5], was presented. Since the tension strength of silicate blocks is negligible, the individual silicate blocks strut is considered to be ineffective in tension. However, the combination of both diagonal struts provides a lateral load-resisting mechanism for the opposite lateral directions of loading.

(b) The equivalent shear wall model consider multi-ribbed wall slab to be equivalent as a shear wall slab by two steps of equivalent[3].The first step, according to equivalent principle of compressive stiffness such as ratio of the elastic module of silicate blocks to concrete, silicate blocks is equivalent as concrete wall. The second step, using equivalent principle of flexural stiffness, the equivalent concrete walls and rib columns are equivalent as an integral concrete shear wall shown in Figure 4(b). The flexural stiffness is calculated according to:

$$E_c I_{eq} = \sum (E_c I_c + E_c I_{w1}) \tag{3-1}$$

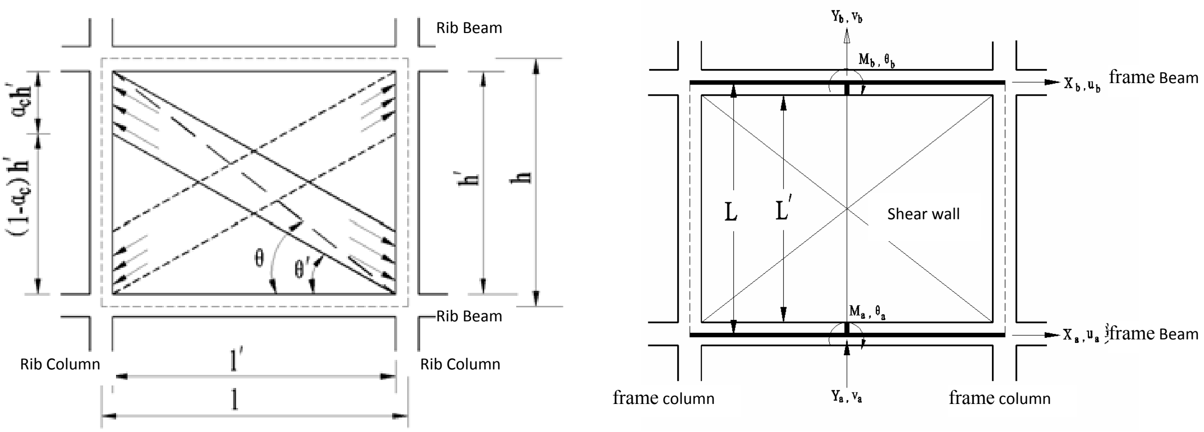
Where,

I_{eq} ——the section moments of inertia of the integral concrete shear wall after second equivalent.

I_{w1} ——the section moments of inertia of the concrete wall after first equivalent.

I_c ——the section moments of inertia of the rib column.

E_c ——the elastic module of concrete.

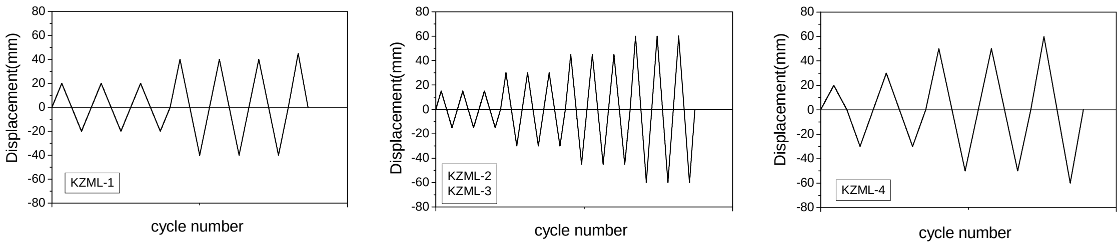


(a) Equivalent strut model (b) Equivalent shear wall model

Figure 4. Simplified models of multi-ribbed wall slab

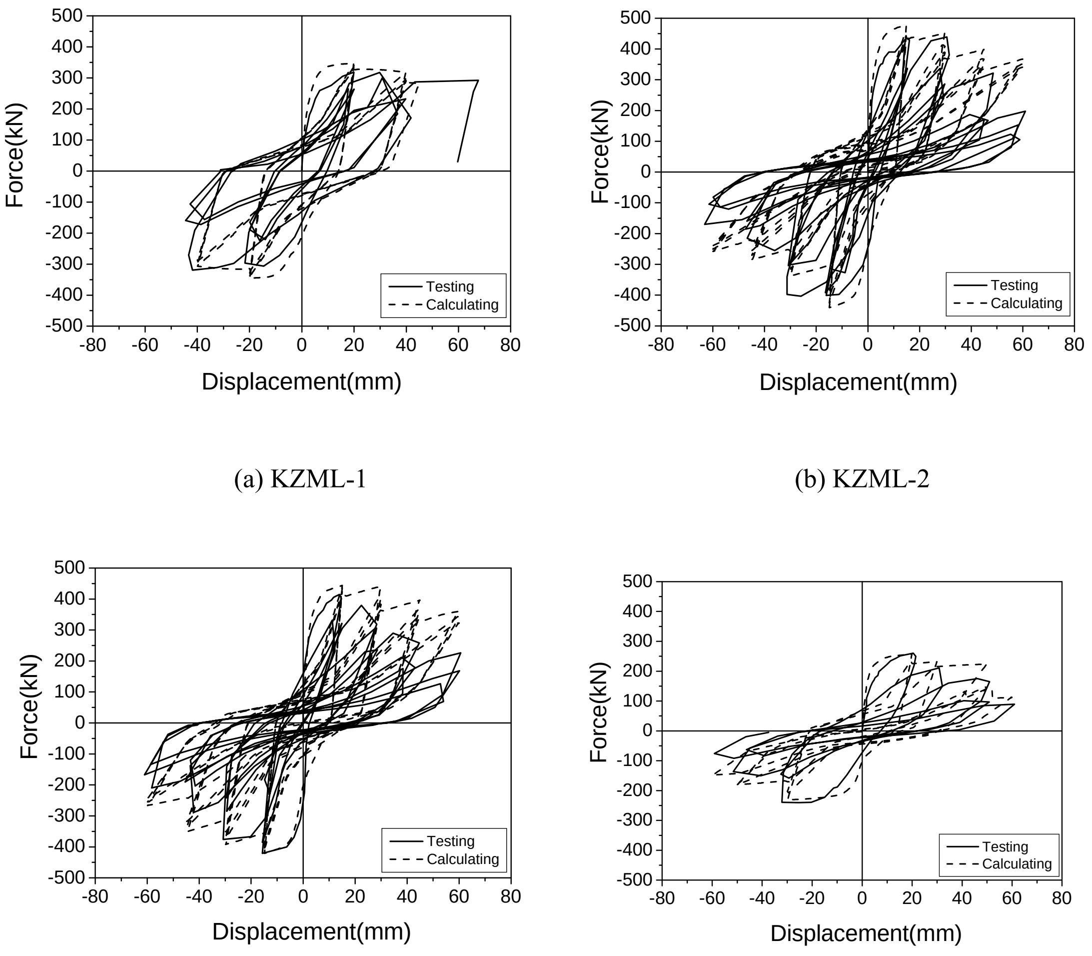
3.2. Pseudo-Static Analysis

The Pseudo-Static analysis of KZML-1~KZML-4 has been carried out by using an equivalent shear wall model. The computing loading system and testing loading system was basically the same shown in Figure 5. The displacement control method was adopted, firstly monotone loading to yielding displacement, secondly cycling under every step with multiples of yielding displacement after yielding.



(a)KZML-1 (b) KZML-2, KZML-3 (c) KZML-4

Figure 5. The computing loading system



(a) KZML-1 (b) KZML-2

(c) KZML-3 (d) KZML-4

Figure 6. Hysteretic curves of the specimens

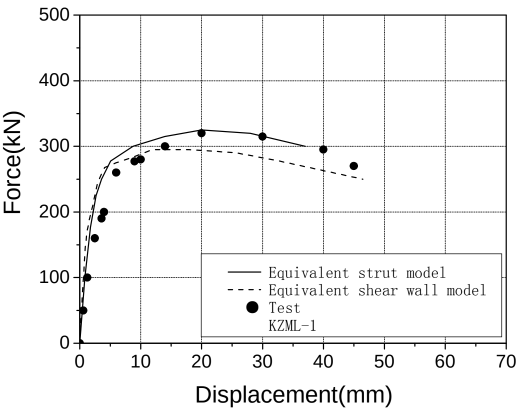
The Pseudo-Static analysis was carried out by using program IDARC2D Version 7.0 [6-7]. Figure 6 give separately the hysteretic curves of the testing and calculating of specimens while its crack, yield, ultimate and failure load-resisting capacity are shown in Table 3.1. The analysis shows that numerical results are in good coincidence with the experimental results, especially in forecast of the values of load carrying capacity at feature points. Therefore, the equivalent shear wall model proposed in the paper is practicable.

Table 3.1. Load Carrying Capacity of Test and Calculating with Pseudo-Static Analysis

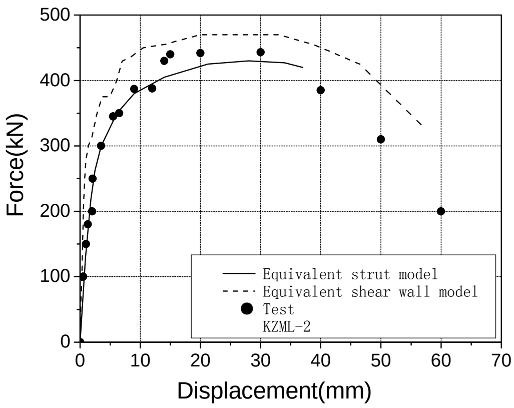
Specimen	Crack		Yield		Ultimate		Failure	
	V_c（kN）		V_y（kN）		V_u（kN）		V_m（kN）	
	Test	Calculating	Test	Calculating	Test	Calculating	Test	Calculating
KZML-1	160	160	280	300	320	330	270	280
KZML-2	180	195	440	435	443	450	370	382
KZML-3	180	180	430	425	430	430	360	366
KZML-4	100	100	260	250	260	250	221	213

3.3. Nonlinear Dynamic Time-History Analysis

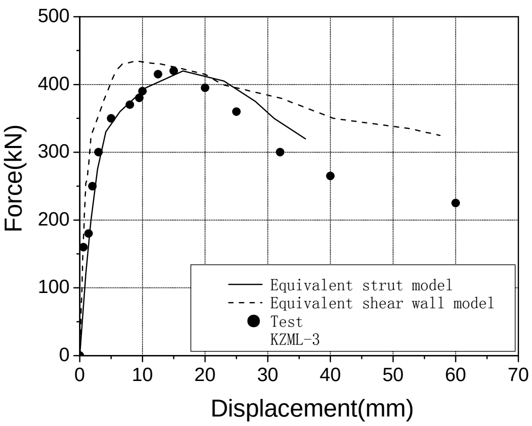
The nonlinear dynamic time-history analysis of KZML-1~KZML-4 under El-Centro wave has been carried out by using separately an equivalent strut model and shear wall model. The computing time of structure is t =30s. The computing time of compression for 1/2 scale models is as $t/\sqrt{2}=21.21s$, and the computing time of compression for 1/3 scale model is as $t/\sqrt{3}=17.32s$. The skeleton curves of shearing force and displacement of test models KZML-1~KZML-4 are shown in Figure 7.



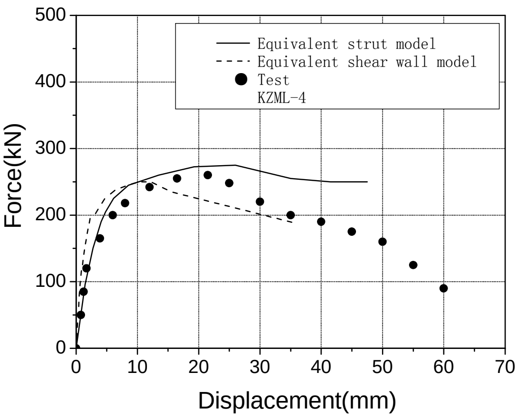
(a)KZML-1



(b) KZML-2



(c)KZML-3



(d) KZML-4

Figure 7. Force-displacement relation curves

Figure 7 (a)~(d) show that the computing results for an equivalent strut model and shear wall model are in good coincidence with the testing results, especially in load-resisting capacity of yielding point and ultimate point (see Table 3.2). Error of calculating yielding load for KZML-1~ KZML-4 using an equivalent strut model are respectively 0.71%,13.64%,8.14% and 5.77% while error of calculating ultimate load are respectively1.56%, 2.93%, 2.33% and 5.77%. Error of calculating yielding load for KZML-1~ KZML-4 using an equivalent shear wall model are respectively 4.29%,10.23%,2.33% and 8.46% while error of calculating ultimate load are respectively 7.81%,6.09%,1.16% and 3.85%. The equivalent strut model and shear wall model proposed above is feasible.

Table 3.2. Load Carrying Capacity of Test and Calculating with Dynamic Analysis

Specimen	Yield（kN）			Ultimate（kN）		
		Strut model	Shear wall model		Strut model	Shear wall model
	Test	Calculating	Calculating	Test	Calculating	Calculating
KZML-1	280	278	268	320	325	295
KZML-2	440	380	395	443	430	470
KZML-3	430	395	420	430	420	435
KZML-4	260	245	238	260	275	250

3.4. Push-Over Analysis

The nonlinear static incremental analysis was carried out using force control. Lateral force distribution was selected using four force distributions available in the program IDARC 2D version 7.0: inverted triangular distribution, uniform distribution, generalized power distribution and modal adaptive distribution [6-7].

a) The inverse triangular distribution considers that the structure is subjected to a linear distribution of the acceleration throughout the building height. The force increment at each step for story “i ” is calculated according to:

$$\Delta F_i = \frac{w_i h_i}{\sum_{l=1}^{N} w_l h_l} \Delta V_b \tag{3-2}$$

where w_i and h_i are the story weight and the story elevation, respectively, and ΔV_b is the increment of the building base shear.

b) The uniform distribution considers a constant distribution of the lateral forces throughout the height of the building, regardless of the story weights. The force increment at each step for story “i” is given by:

$$\Delta F_i = \frac{\Delta V_b}{N} \tag{3-3}$$

Where ΔV_b is the increment in the base shear of the structure, and N is the total number of stories in the building.

c) The generalized power distribution was introduced to consider different variation of the story accelerations with the story elevation. This distribution was introduced to capture different modes of deformation, and the influence of higher modes in the response. The force increment at floor “i” is calculated according to:

$$\Delta F_i = \frac{w_i h_i^k}{\sum_{l=1}^{N} w_l h_l^k} \Delta V_b \tag{3-4}$$

where k is the parameter that controls the shape of the force distribution. The recommended value for k may be calculated as a function of the fundamental period of the structure (T):

$k = 1.0$ （$T \leq 0.5$s)

$$k = 1.0 + \frac{T-0.5}{2.5-0.5} \quad (0.5\text{s}< T < 2.5\text{s})$$

$$k = 2.0 \quad (T \geq 2.5\text{s}),$$

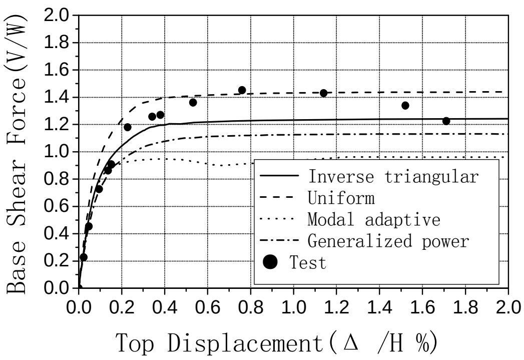
Nevertheless, any value for k may be used to consider different acceleration profiles. Note that $k=0$ produces a constant variation of the acceleration, while $k=1.0$ produces a linear variation (inverted triangle distribution), and $k=2.0$ yields a parabolic distribution of story accelerations.

d) The modal adaptive distribution differs significantly from all the previous ones in that the story force increments are not constant. The modal adaptive distribution was developed (Reinhorn, 1996, 1997) to capture the changes in the distribution of lateral forces. Instead of a polynomial distribution, the "instantaneous" mode-shapes of the structure are considered. Since the inelastic response of the structure will change the stiffness matrix, the mode shapes will also be affected, and a distribution proportional to the mode shapes will capture this change. The increment in the force distribution is calculated according to:

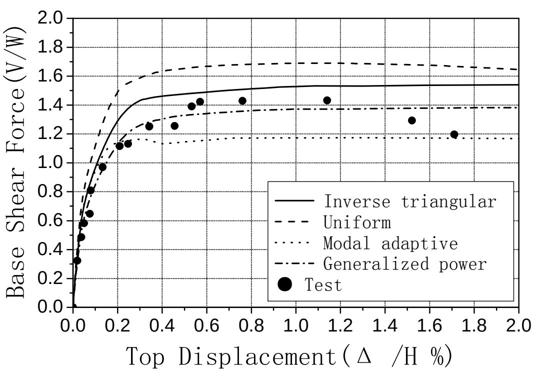
$$\Delta F_i = \frac{w_i [\sum_{j=1}^{m} (\Phi_{ij}\Gamma_j)^2]^{1/2}}{\sum_{l}^{n} w_l [\sum_{j=1}^{m} (\Phi_{ij}\Gamma_j)^2]^{1/2}} V_b - F_i^{old} \tag{3-5}$$

Where Φ_{ij} is the value of "instantaneous" mode shape "j" at story "i", Γ_j is the modal participation factor for mode "j", V_b is the new base shear of the structure, and F_i^{old} is the force at floor "i" in the previous loading step.

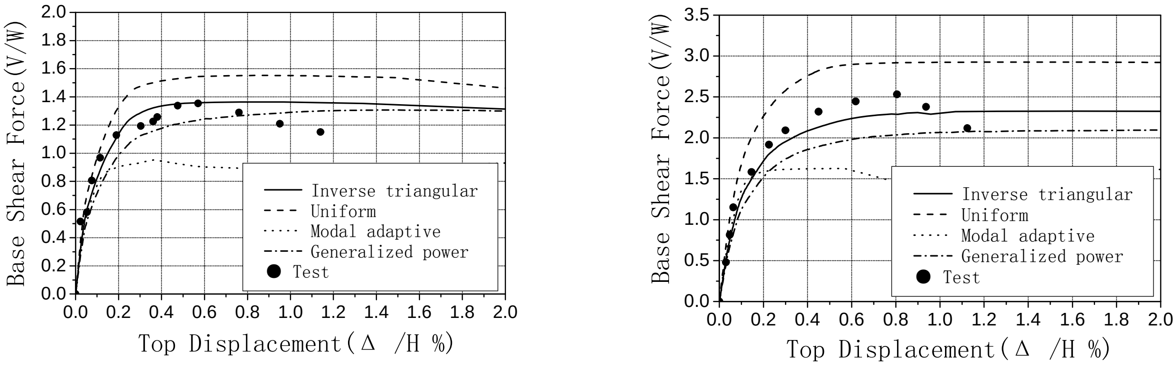
The nonlinear static incremental analysis of KZML-1~KZML-4 under four force distributions has been carried out by using an equivalent shear wall model. The push-over curves as well as test skeleton curves of KZML-1~KZML-4 are shown in Figure 8. From Figure 8 (a)~(d) it can been seen that push-over curves of four force distributions are in good coincidence with the experimental results, which the uniform distribution gives the upper limit of push-over curves and the modal adaptive distribution gives the lower limit of push-over curves.



(a) KZML-1



(b) KZML-2



(c) KZML-3 (d) KZML-4

Figure 8. The push-over curves of KZML-1~KZML-4

4. CONCLUSIONS

The pseudo-static tests and nonlinear numerical modeling show that the multi-ribbed slab structure has a good earthquake-collapse resistant capacity. The pseudo-static analysis, nonlinear dynamic time -history analysis and push-over analysis show that numerical calculating results are in good coincidence with the experimental results, especially in forecast of the values of the load carrying capacity at feature points, which indicated that the equivalent strut model and shear wall model proposed in the paper are practicable.

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