RESEARCH ON THE SEISMIC DESIGN OF TRANSFER-STOREY STRUCTURES BASED ON PHILOSOPHY OF CAPACITY DESIGN UNDER SEVERE EARTHQUAKE

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ABSTRACT :

Both theoretical analysis and engineering applications of the transfer-storey structures are mostly limited in elastic stage under frequent earthquakes; the structural behavior after yielding under severe earthquake need to be studied by elastic-plastic analysis. And there is no specific and quantitative method to ensure that transfer structures can "resist severe earthquake without collapse". In this paper, the philosophy of capacity design is introduced into the design of transfer-storey structures and a practical design method for transfer structures under severe earthquake is presented. Through push-over analysis, various limit states of coupled shear walls are disclosed. Parameters that influence significantly the structure behavior such as integer coefficient of coupled shear walls, reinforcement ratio of coupling beams and height-width ratio of shear walls are highlighted. The shear overstrength of the coupling beams under the ideal limit state is reduced with reduction coefficient *K*. Overstrength of shear walls is derived by ultimate state analysis, which considers the axial forces increment of shear walls caused by shear overstrength of coupling beams. On this basis, mechanical characteristics and capacity based seismic design method of transfer storey under severe earthquake are studied by elastic-plastic analysis. A series of nonlinear dynamic time-history analysis has been completed, which considers the influence of varied parameters including the stiffness and mass of transfer structures, seismic protected intensity and the position of transfer storey. Moreover, the simplified formulas and the detail procedure for capacity design of transfer structures are presented.

KEYWORDS: philosophy of capacity design, design of transfer-storey, structural overstrength, push-over analysis, nonlinear dynamic time-history analysis, severe earthquake

1. INTRODUCTION

In china, research and applications of transfer-storey structures begin in 1970s and rapidly developed in the past twenty years. At present, both theoretical analysis and engineering applications of the transfer-storey structures are mostly limited in elastic stage under frequent earthquakes; the structural behavior after yielding under severe earthquake need to be studied by elastic-plastic analysis. The seismic assessment methodologies suggested by Chinese codes or the (G+βE) method used in practical engineering which enlarged horizontal earthquake action with coefficient β can't quantitatively ensure that transfer structures, such as transfer primary and secondary girders, transfer secondary and secondary girders, transfer cantilever beams, can "resist severe earthquake without collapse".

In this paper, the philosophy of capacity design is introduced into the design of transfer-storey structures. Pushover analysis is completed to obtain the reduction coefficient K for the strength of coupling beams. Overstrength of shear walls is derived by ultimate state analysis, which considers the axial forces increment of shear walls caused by shear overstrength of coupling beams. On this basis, mechanical characteristics and capacity based seismic design method of transfer storey under severe earthquake are studied by elastic-plastic analysis. A series of nonlinear dynamic time-history analysis has been completed, which considers the influence of varied parameters including the stiffness and mass of transfer structures, seismic protected intensity and the position of transfer storey. Moreover, the simplified formulas and the detail procedure for capacity design of transfer structures are presented.

2. DESIGN PRINCIPLE BASED ON PHLIOSOPHY OFCAPACITY DESIGN

The design philosophy based on capacity design was proposed by professor R. Park and professor T. Paulay [1], the key idea of which is that the ideal locations of plastic hinges are chosen and suitably detailed to ensure the ductility capacity of seismic-resistant structure, and other adverse plastic hinges or failure mechanisms (brittle failures) should be avoided by providing sufficient reserve strength capacity (considering overstrength factor).

According to the philosophy of capacity design, the first step of designing a ductile earthquake-resistant structure successfully is to choose a reasonable plastic mechanism. For instance, a plastic mechanism chosen for a transfer-storey structure is shown in Fig.1 [2, 3], which is on the basis of the strong transfer and weak upper structure theory.

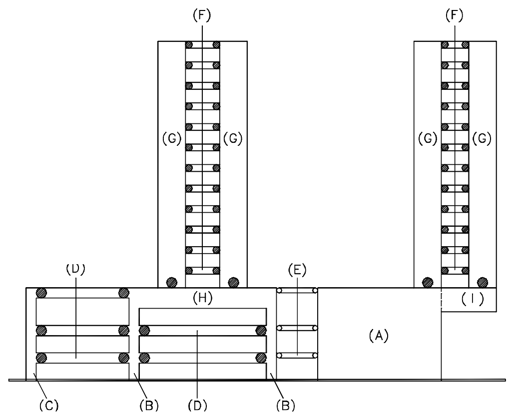


Fig.1 Plastic Mechanism According to Strong Transfer and Weak Upper Structure Theory

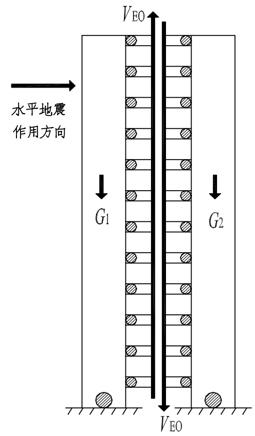


Fig.2 Shear overstrength of the coupling beams VEO

The potential plastic hinges of the transfer-storey structure shown in Fig.1 may be formed at the ends of frame beams (D), the ends of upper coupling beams (F) and the bottom of upper shear walls (G), which can provide ductility to the whole structure, while other structural elements are still in elastic stage throughout seismic excitation. Therefore, the ideal energy-dissipating mechanisms of transfer-storey structures should be that all the coupling beams and frame beams reach their ultimate plastic states, and plastic hinges appear at the bottom of upper shear walls, while other structural elements are still in elastic stage. Under this ideal state, by analyzing ultimate bearing capacity of the shear walls [2, 4] and considering adequately the overstrength factors of upper coupling beams (F) and upper shear walls (G), we obtain the reactions of upper structures at ultimate limit state, which are then acted on the transfer structure. Consequently, the key members in capacity design, such as ground shear walls (A), supporting columns (B), transfer girders (H) and cantilever transfer beams (I) are determined for bearing capacity at the elastic state.

3. THE SHEAR OVERSTRENGTH OF COUPLING BEAMS

The assumption that the upper structure is under ideal ultimate state is real when it is designed based on ‘strong walls and weak coupling beams’ which follows the capacity design method. But in practical engineering, it is designed based on domestic design codes. Since detail requirements and human factors will cause to uncertain overstrength factors, the ideal ultimate state assumption may be false.

The size and number of openings of coupled shear walls affect mechanic properties, distribution of stress, deformation states and failure modes of the structure. It means that shear overstrength V_{EO} of coupling beams has grate effect on coupled shear walls. Obviously, it is conservatively if $V_{EO} = \sum_{1}^{n} V_{EO,i}$, where $V_{EO,i}$ is shear overstrength of each beam (Fig. 2). Ref. [5] pointed that plastic hinges will not be formed all together (or some plastic hinges of coupling beam whose deformation is too large will be out of work), so the V_{EO} should be reduced as following:

$$V_{EO} = (1 - \frac{n}{80})\sum_{1}^{n} V_{EO,i} \tag{3.1}$$

where n is number of upper coupling beams, and n⩽20 (if n>20, then n=20).

Although the number of upper coupling beams or the height-width ratio of upper shear walls was considered in this equation, it is quite different with practical condition. Hereinafter, various ultimate states of coupled shear walls will be disclosed through pushover analysis. Reduction coefficient K is introduced into reducing overstrenght of coupling beams at ultimate limit state.

3.1. Material Overstrength at Ultimate Limit State

In capacity design, the material overstrength should be the statistical average value or the test value of material. As the test values can't be obtained in structural analysis phase, it is reasonable to choose the most representative statistical average values of material as the performance indexes according to the grades of reinforcement and concrete which are decided by designer. On the other hand, according to Chinese current design codes the restraining edge members or constructional edge members must be set in high-rise shear wall structures. In this way, the compressive area of shear walls are restrained very well and the ultimate compressive strain ε_{cu} usually ranges from 0.012 to 0.050, which exceeds 4 to 6 times of normal estimated value of unconstrained concrete [5], moreover the curvature ductility of sections in shear walls is sufficient [6]. It is expected that after strain hardening the strength of reinforcements in tensile area of shear walls will reach the average value of ultimate tensile strength f_{tm} . With reference to [7], the values of average yielding strength f_{ym} and average ultimate tensile strength f_{tm} of reinforced bars are listed in table 3.1. The compressive strength of concrete affects the height of compressive area in members under compression bending or pure bending, but has little effect on the flexure overstrength. Therefore, the compressive strength of concrete may be the average compressive strength f_{cm} in Ref. [7] table 3.2, neglecting the strength increment of confined concrete (see Table 3.2).

Table 3.1 Average tension strength of reinforcement

Types	f_{ym} (MPa)	f_{um} (MPa)
HPB235	275	385
HRB335	390	545
HRB400	470	660

Table 3.2 Average strength of concrete

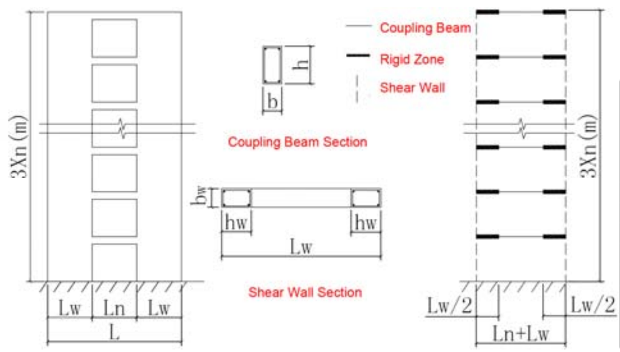
Types of strength	Concrete strength grade												
	C20	C25	C30	C35	C40	C45	C50	C55	C60	C65	C70	C75	C80
f_{cm}	19.0	22.7	26.1	29.8	33.3	36.9	39.5	43.3	46.1	49.7	53.3	56.7	60.1
f_{tm}	2.19	2.41	2.61	2.80	2.98	3.13	3.23	3.34	3.41	3.50	3.58	3.65	3.72

3.2. Push-over Analysis

3.2.1 Modeling approach

Non-linear analysis of coupled shear walls have been performed using IDARD-2D6.0 [8], which is a two dimensional analysis program developed by Buffalo University in New York, U.S.A. According to assumptions of IDARC-2D, the computing models are developed as shown in Fig. 3. The number of stories *n* is 10, 15 and 20 respectively (storey height is 3m). The linear stiffness of wall limbs is larger than 5 times of the linear stiffness of coupling beams (ensuring that wall limbs won't become wall-frame column) [9]. The integer coefficient A [10] of coupled shear walls is no more than 10, that is $A < 10$. The value of A is changing with the section of coupling beams and storey numbers *n*. The section of wall limbs is 200mm x 2000mm, 200mm x 3000mm and 200mm x 4000mm respectively. The section of coupling beams ρ_b is changing from 200mm x 200mm to 200mm x 800mm. On this basis, reinforcement ratios of coupling beams and wall limbs are considered. The reinforcement ratio of coupling beams is 0.2, 0.3, 0.4 and 0.5 respectively. The reinforcement ratio of wall limbs is divided into two types. Type I: Reinforcement ratio of boundary columns is1.0 and reinforcement ratio of wall panels is 0.25. Type II: Reinforcement ratio of boundary columns is 1.1 and reinforcement ratio of wall panels is 0.4. The concrete strength grade is C30; the type of reinforcement is HRB400. Nonlinear static analysis of coupled shear walls is completed considering different lateral force patterns, and the average strength value of material is used. Chinese codes limit the axial compression ratio at 0.5 in case of the 1st seismic fortification grade (the earthquake intensity is 7 or 8 degree) and 0.6 in case of the 2nd seismic fortification grade. The axial force of shear walls under gravity load in this model is deduced by the axial compression ratio. Then each joint mass is deduced.

The shear wall computing models in IDARC-2D neglect that the moment-curvature curve effected by axial force variation. But shear forces of coupling beams will change axial forces of wall limbs, and then influence the moment-curvature curve. Here are two ways to reduce the effects of shear wall models on analysis results. One way is restricting the computing models for reducing the effects of coupling beams on axial force of wall limbs. It is find that the skeletons of moment-curvature curve of wall limbs are near when axial forces change within a certain limit ($\pm 20\%$). The other way is adjusting the initial axial forces of wall limbs properly. Firstly, calculate the shear overstrength of coupling beams preliminarily follow equation (3.1); then add a part of the shear overstrength of coupling beams to initial axial forces, making the rest of the shear overstrength changing axial forces within 20%. The computing parameters of 10-storey models are listed in table 3.3.



（a）actual model （b）computing model
Fig.3 Actual model and computing model

Table 3.3 Computing parameters of 10-storey models

number	n	b_w (m)	h_w (m)	L_w (m)	b (m)	h (m)	L (m)	A	initial axial force (kN)	ρ_b (%)	V_{EO} (kN)	Adjusted initial axial force (kN)	
												left limbs	right limbs
A0	10	0.2	0.4	3	0	0	9	0	4290	0	0	4290	4290
A1	10	0.2	0.4	3	0.2	0.2	9	0.83	4290	0.2	43	4290	4290
A2	10	0.2	0.4	3	0.2	0.3	9	1.48	4290	0.2	111	4290	4290
A3	10	0.2	0.4	3	0.2	0.4	9	2.21	4290	0.2	209	4290	4290
A4	10	0.2	0.4	3	0.2	0.5	9	2.98	4290	0.2	339	4290	4290
A5	10	0.2	0.4	3	0.2	0.6	9	3.78	4290	0.2	499	4290	4290
A6	10	0.2	0.4	3	0.2	0.7	9	4.6	4290	0.2	690	4290	4290
A7	10	0.2	0.4	3	0.2	0.8	9	5.42	4290	0.2	912	4423	4357

3.2.2 Results and discussion.

From push-over analysis results, it is observed that failures of coupled shear walls don't always develop according to ideal ultimate failure states. Failure states can be divided into 3 types:
(I) Coupling beams failed, and wall limbs took on all of the lateral force.
(II) Coupling beams yielded, and then wall limbs failed in flexure and lost bearing capacity. This type can be subdivided into two types, plastic hinges developed fully (IIA) and plastic hinges did not develop fully (IIB).
(III) Coupling beams did not yield, and wall limbs failed in flexure and then lost bearing capacity.

Mainly factors that influence the ultimate state of coupled shear walls are as following:
(1) The integrity of coupled shear walls, which is represented by integer coefficient A. As A increasing, lateral force resisting capacity of coupled shear walls increase, and the ultimate failure states change from type (I) to type (II), and then type (III).
(2) The reinforcement ratio of coupling beams ρ_b, which mainly affect coupled shear walls with weak integrity. With the increase of ρ_b, the ultimate failure states of coupled shear walls change, and bearing capacity enhance in some degree. However, for coupled shear walls with strong integrity, reinforcement ratios of coupling beams make little effect.
(3) The number of stories *n*. Actually it is height-width ratio of wall limbs $\chi = H / L_w$. Despite the integer coefficient has considered the influence of storey number, when the integer coefficient and reinforcement ratio of coupling beams remain unchanged, plastic hinges of coupling beams develop more fully in structures with larger number of stories. The reason is that bending deformation becomes primary deformation when number of stories *n* becomes larger and then height-width ratio of wall limbs χ becomes larger. And when the bending deformation is large, the rotation of coupling beams is large; further the plastic hinges of coupling beams

develop fully.

3.3. Strength Reduction Coefficient K of Coupling Beams

The integer coefficient A, the reinforcement ratio of coupling beams ρ_b and the number of stories n influence the ultimate failure states of coupled shear walls and the plastic deformation of coupling beams. So they influence the variation of axial force of wall limbs. A reduction coefficient is introduced to reduce the shear overstrength of coupling beams, named as strength reduction coefficient K of coupling beams. Through inductive analysis, the reduction formulation of coupling beams is induced:

$$K = 10^{-4}(3\chi + 15)\rho_b^{-1}A^{-0.8} \tag{3.2}$$

For checking the correctness of this formulation, recalculate the reduction coefficients K of models using the above formulation. Comparing the results with the nonlinear analysis results, it is observed that they are similar (see Table 3.4). So it can be proved that the induced formulation is correct. In addition, the ultimate failure state of coupled shear walls can be determined by this formulation (see Table 3.5).

Table 3.4 Failure type and comparison of the reduction factor

number	($L_n = 2000mm$)							
	A	ρ_b(%)	vibration of axial force of wall limbs caused by coupling beams (kN)	V_{EO} (kN)	$\sum_1^n V_{EO,i}$ (kN)	real reduction factor	K	failure type
A0	0.00	0	0	0	0	——	—	I
A1	1.25	0.2	0	65	74	——	1.88	I
A2	2.19	0.2	0	166	190	——	1.20	I
A3	3.20	0.2	269	314	359	0.75	0.89	IIA
A4	4.24	0.2	355	508	581	0.61	0.71	IIB
A5	5.28	0.2	453	748	855	0.53	0.59	IIB
A6	6.29	0.2	565	1035	1183	0.48	0.52	IIB
A7	7.26	0.2	687	1368	1563	0.44	0.46	IIB

Table 3.5 Failure types under ultimate state determined by coefficient K

K	Failure types
$K \ge 1$	I
$1 > K \ge 0.75$	IIA
$0.75 > K \ge 0.4$	IIB
$K < 0.4$	III

4. OVERSTRENGTH OF UPPER SHEAR WALLS

Take coupled straight shear wall for example, the ultimate failure state and overstrength of upper shear walls are illustrated as following.

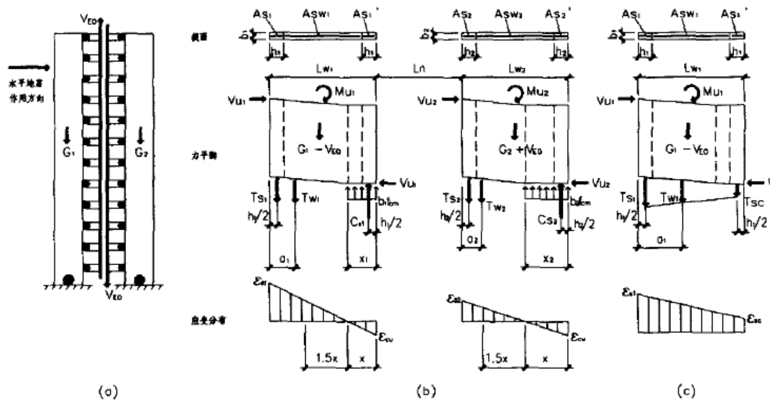


Fig. 4 Ultimate state of coupled straight shear walls

Fig. 4 shows the ultimate state of coupled straight shear walls. The shear overstrength V_{EO} of coupling beams act on wall limbs, leading to axial force of a wall limb increase and the other wall limb decrease. L_w is the shear wall width. b is the shear wall thickness. h is the boundary column height. G is the representative value of gravity load. M_u is the flexural overstrength of shear walls. T_s is the resultant force of longitudinal reinforcement of tensile boundary column in shear wall. T_w is the resultant force of effective tensile distributing reinforcement in web of shear wall. C_s is the resultant force of longitudinal reinforcement of compressive boundary column in shear wall. V_u is the designed shear force of shear walls corresponding to flexural overstrength. A_s ,

A_{sw} and $A_s^{'}$ is the reinforcement area of tensile boundary column, wall segment and compressive boundary column in shear wall, respectively. a is the distance from T_s to edge. x is the height of ultimate compressive area. The subscript 1 and 2 corresponding to left wall limb and right wall limb respectively. According to the equilibrium condition of wall limbs the M_u can be calculated. Then the overstrength factor of upper shear wall structure and the shear force of upper shear walls corresponding to flexural overstrength are derived as following:

$$\phi_{o,w} = \frac{M_{u1} + M_{u2} + V_{EO}(\frac{L_{w1}}{2} + \frac{L_{w2}}{2} + L_n)}{M_E}, \quad V_{wi,EO} = \phi_{o,w} \cdot V_E, \quad V_{wi,EO} = \phi_{o,w} \cdot V_E \cdot (\frac{M_{ui}}{M_{u1} + M_{u2}})(i = 1,2) \quad (4.1)$$

where, M_E is the overturning moment of coupled shear walls calculated by mode-superposition response spectrum method or elastic time-history analysis. V_E is the horizontal shear force corresponding to M_E. $V_{w,EO}$ is the estimated value of designed shear force of shear walls corresponding to flexural overstrength.

5. DESIGN FORMULATION FOR TRANSFER MEMBERS BASED ON PHILOSOPHY OF CAPACITY DESIGN

The flexural overstrength and corresponding shear forces of shear walls are derived above through the limit analysis. We can conclude that there are four effects which will influence the design of the transfer-storey structures: (1) The effect of transfer structures S_{Mu} under flexural overstrength M_u of the upper structures; (2) The effect of transfer structures S_G under gravity loads of themselves; (3) The effect of transfer structures S_{Vu} under the shear force V_u in correspondence with the flexural overstrength of the upper structures; (4) The effect of the transfer structures S_{Er} under the severe earthquake. Therefore the capacity-based design formulations for transfer members are given as:

$$S_0 = S_{Mu} + S_G + S_{Vu} + S_{Er} \quad (5.1)$$

$$S_0 \le R_k \quad (5.2)$$

where S_0 is the standard value of transfer members' combinational internal force based on philosophy of capacity design, including combinational moment, axial force and shear force;

R_k is the standard value of members' bearing capacity based on current Chinese codes. And strength of material is standard value, not considering anti-seismic adjusting factor γ_{RE}.

In practical engineering, "accurate" calculation means to do the non-linear analysis which brings great difficulties for extensive application. By non-linear analysis [2,3,4], it has been found that S_{Er} is only a small portion of S_0. Therefore, it is reasonable and practical to give the capacity design formulation of transfer members as following, which considered influencing factors under severe earthquake including mass and stiffness of transfer storey, earthquake intensity, and transfer storey location [3]:

$$S_0 = S_{Mu} + S_G + \alpha\beta S_{Vu} \quad (5.3)$$

where α is an amplification coefficient of transfer-storey structures under severe earthquake related to transfer storey stiffness; β is an amplification coefficient of transfer-storey structures under severe earthquake related to transfer storey mass.Under severe earthquake, elastoplastic analyses of plentiful engineering examples which designed according to current Chinese codes at 7 and 8 degree earthquake intensity have been completed to induce the suggested values of α and β.Actual strong ground motion records [11] have been selected for elastoplastic analysis of structures. The selection of ground motions considered influences of structural properties, site characters, near seismic effect and teleseismic effect. The analyses are also performed using IDARC-2D6.0 [8].

5.1. Amplification Coefficient β of Transfer-storey Structures under Severe Earthquake Related to Transfer Storey Mass

In order to obtain the earthquake action on transfer structures considering the influence of different transfer storey mass among transfer truss, transfer beams and transfer plates (presented as representative value of gravity load every square meter of the transfer storey), plenty of engineering examples at 7 and 8 degree earthquake intensity have been analyzed. The representative values of gravity load every square meter of the transfer storey G_t are 23.2 kN/m^2, 46.4 kN/m^2, 69.6 kN/m^2, and 92.8 kN/m^2 respectively. Through elastoplastic time history analyses in each condition, maximum floor shear V_i of floor i, maximum seismic responses of structures F_i, and shear force V_u corresponding to flexural overstrength of upper shear walls are obtained. Then internal forces of members in transfer storey and below under various working conditions can be calculated.According to equation $S_{Vu} + S_{Er} = \alpha\beta S_{Vu}$, we assume that $\alpha = 1$ in transfer beams condition, then $\beta = 1 + S_{Er} / S_{Vu}$. Combined with above analyses results, the amplification coefficient β of transfer-storey structures under severe earthquake related to transfer storey mass will be obtained by analyzing structures in 7 degree earthquake intensity zone (0.1g) with transfer storey located in floor 5 and structures in 8 degree earthquake intensity zone (0.2g) with transfer storey located in floor 3 (see Fig. 5, Fig. 6)

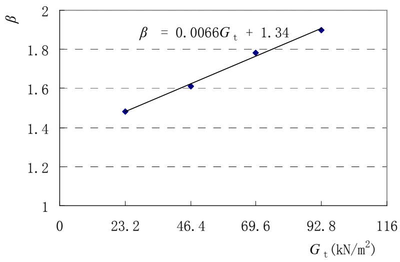


Fig.5 Gt-β curves under earthquake intensity

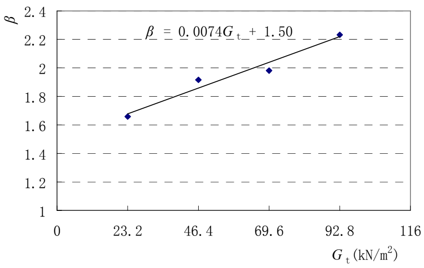


Fig.6 Gt-β curves under earthquake intensity

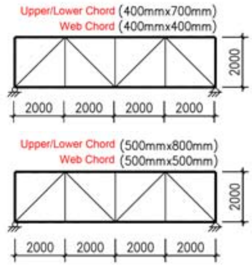


Fig.7 Transfer truss

5.2. Amplification Coefficient α of Transfer-storey Structures under Severe Earthquake Related to Transfer Storey Stiffness

In order to obtain the earthquake action on transfer structures considering the influence of different transfer storey stiffness among transfer truss, transfer beams and transfer plates, on the premise of other conditions (such as transfer storey mass and damping) unchanged, elastoplastic time history analyses have been conducted. In transfer beams condition, the sections of transfer beams are 1000mmx2000mmm, 1000mmx2300mm and 1000mmx2600mm respectively. In transfer plate condition, the thicknesses of plates are 2000mm, 2300mm and 2600mm respectively. In transfer truss condition, forms shown in Fig. 7 are considered.

Through time history analyses, it is found that different transfer member sections of same transfer form structures in 7 degree earthquake intensity zone have little influence on base shear of structures and earthquake response of transfer storey and each below floor (approximately changed within 5%). The change of structure internal forces in 8 degree earthquake intensity zone is the same as that in 7 degree which changed within 5%.

Difference of transfer form has a great effect on earthquake responds of structures in 7 degree earthquake intensity zone. Regarding transfer beam structure as reference object, earthquake responds of transfer plate are similar to transfer beam. However, in transfer truss condition, the earthquake responds of transfer storey and all floors below transfer storey are smaller than transfer beam about 10%~15% on average. Now transfer truss structure is regarded as reference object. Assume $\alpha = 1.0$ in transfer truss condition, then $\alpha = 1.05$ in transfer beam and transfer plate conditions.

5.3. Amplification Coefficient of Transfer-storey Structures under Severe Earthquake

The amplification coefficient α and β are obtained from above analyses of structures in 7 degree earthquake intensity zone (0.1g) with transfer storey located in floor 5 and structures in 8 degree earthquake intensity zone (0.2g)with transfer storey located in floor 3. Similarly, amplification coefficient α and β will be obtained by analyzing all kinds of transfer structures in different earthquake intensity zone and with different transfer storey locations. For engineering applications, it is convenient that the results of α and β are simplified by induction. It can be observed that coefficient α is influenced greatly by stiffness of transfer storey (shown as transfer forms) but little by earthquake intensity and transfer storey location. Earthquake intensity and transfer storey location will be considered in coefficient β. So it can be induced that α is only related to transfer forms (see Table 5.1). β is related to representative value of gravity load every square meter of the transfer storey, earthquake intensity and location of transfer storey:

$$\beta = 0.007G_t + \beta_0 \tag{5.4}$$

where G_t is representative value of gravity load every square meter of the transfer storey, and $G_t = D_t + L_t / 2$, where D_t is average value of dead load every square meter of the transfer storey, L_t is average value of live load every square meter of the transfer storey. The unit of G_t, D_t and L_t is kN/m^2. β_0 is initial amplification coefficient of transfer-storey structures under severe earthquake related to transfer storey mass, which is influenced by earthquake intensity and location of transfer storey (see Table 5.2).

Table5.1 Amplification factor α according to severe earthquake reaction related to transfer storey stiffness

α	transfer types
1.05	transfer beam, transfer plate, transfer box
1	transfer truss

Table5.2 Initial enlarged factor β_0 according to severe earthquake reaction elated to transfer storey mass to transfer storey stiffness

earthquake intensity	transfer storey location				
	floor 1	floor 2	floor 3	floor 4	floor 5
6 degree 0.05g	1.10	1.16	1.20	1.24	1.28
7 degree 0.10g	1.18	1.22	1.26	1.30	1.34
7 degree 0.15g	1.30	1.34	1.38	1.42	——
8 degree 0.20g	1.40	1.44	1.50	——	——
8 degree 0.30g	1.44	1.48	——	——	——
9 degree 0.40g	1.48	——	——	——	——

6. DESIGN PROCEDURE FOR TRANSFER MEMBERS BASED ON PHILOSOPHY OF CAPACITY DESIGN

6.1. Detail Procedure for Transfer Members Based on Philosophy of Capacity Design

The detail procedure for transfer members based on philosophy of capacity design is presented as following:
(1) Design according to current codes.
(2) Calculate shear force of upper structures follow Ref. [11]. The flexure overstrength M_u of shear wall in each direction, overstrength factor $\psi_{0,w} = M_u / M$ and design value of shear force $V_u = \psi_{0,w}V$ corresponding to total flexure overstrength should be obtained. The strength value of material is average value.
(3) Look up Table 5.1 according to the transfer type for α and look up Table 5.2 according to the earthquake intensity and transfer storey location for β_0. Then substitute β_0 and G_t into equation (5.4), and β is obtained.
(4) M_u and $\alpha\beta V_u$ corresponding to direction +X, -X, +Y, -Y respectively are acted on transfer storey, and gravity load of transfer storey was considered (partial coefficients of dead load and live load are 1.0, strength value of materials is standard value). Then S_0 and reinforcement requirement in each direction are obtained through equation (5.3).
(5) Check that transfer structure can satisfy equation (5.2). If don't satisfy, then modify transfer structure and check again. If the structure has little change, then come to step (4); if the structure has great change, then come to step (1).

This procedure may be too complicated at the preliminary design stage. In general conditions (excluding high earthquake intensity and high transfer storey location), $S_{Vu} + S_{Er}$ has little effect on transfer member design. And it is acceptable that $S_0 = 1.1(S_{Mu} + S_G)$ in capacity design. Therefore, relative transfer members can be isolated and designed individually. Moreover, according to engineering experience the key overstrength direction can be judged.

6.2. Comparison between Capacity Design Method and Other Methods

For partial span wall on transfer beams, the $G + \beta E$ method used in engineering has much more capacity demand than other methods in the respect of flexural bearing capacity, and the capacity design method presented in this paper has more capacity demand than the method suggested by Chinese codes. In the respect of shear capacity, the $G+\beta E$ method has more capacity demand than other methods, and the capacity design method has little less capacity demand than the current Chinese code method.

For supporting columns and ground shear walls, the $G+\beta E$ method and capacity design method have much more shear capacity demand than the current Chinese code method. But checking the corresponding shear stress ratio it is found that shear capacity demand is easy to be satisfied in any method.In the respect of capacity demand of transfer storey structure, the $G+\beta E$ method has greatest capacity demand; the current Chinese code method has the least capacity demand; the capacity design method is intermediate. Moreover, the capacity design method, which has little more safety reserve than the current Chinese method, is economic, reasonable and clear in concept.

7. CONCLUSIONS

This paper has provided a reasonable and practical capacity design method for transfer storey structures under severe earthquake. The major conclusions of this study are as following:
(1) Various limit states of coupled shear walls are disclosed through push-over analysis.
(2) The reduction coefficient K is induced by analysis of parameters including integer coefficient of coupled shear walls, reinforcement ratio of coupling beams and height-width ratio of shear walls.
(3) Overstrength of shear walls is derived by ultimate state analysis, which considers the axial forces increment of shear walls caused by shear overstrength of coupling beams.
(4) The simplified formulas and the detail procedure for capacity design of transfer structures are presented. A series of nonlinear dynamic time-history analysis has been completed, which considers the influence of varied parameters including the stiffness and mass of transfer structures, seismic protected intensity and the position of transfer storey.

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