Seventh Canadian Masonry Symposium
McMaster University
Hamilton, Ontario
June 4-7, 1995

A STUDY FOR BEHAVIOUR AND STRENGTH OF PERFORATED BRICK MASONRY WALLS SUPPORTED ON FRAME UNDER UNIFORM LOADING

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ABSTRACT

The experimental results on six specimens of the perforated brick masonry walls supported on frame are given. The influence of height-span retio of walls H_w/l_o, beam-column rigid ratio of frame K_{mb}/K_{mc} and percentage of longitudinal bars of beam $\rho = A_s/bh_o$ on the behaviour and strength are discussed. The plastic limit analyses for bearing capacity of walls-framecomposite structures are carried out and the formulas of their flexural and shear strength, which tally with the test results, are presented. The suggestions of design are put forward for the masonry walls on frame.

INTRODUCTION

Themasonry walls supported on frame arecomposed ofthereinforced concrete frameand themasonry walls supportedonthem.Theyare widely used in the multistoried masonry buildings, for example,thebuildings, in whichshopson thegroundfloor anddwelling houseon others. In the fifties of 20th century the elastic analyses for wall-beams have been, carried out (Jemochkin, 1960).The study forthe composite actionof thebrickpanel walls supported onreinforced concretebeams(Wood,1957) and the experimentalstudyof masonry walls on beams (Resenhaupt, 1962) havebeen done. Since the seventiesof20th century, theexperimental research onabout256specemens ofsimple supporting wall-beams and 15

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specemens of continuous wall-beams, andalotofthe finite element analyses ofwall-beams have been done by the Research Group for wall-beams ofChina, including worksof the authors (Feng et al, 1989), (Gong et al, 1988), (Gong,1989). The behaviourand modes of failure ofthe simple wall-beams were discussed and the formulas of flexural and shear strength of wall-beams and local compressive strength of masonry above supporting beam at supports were gaven. Therefor the design of wall-beams has beencoveredby the Code for Masonry Structures (GBJ3-88) in China (Tanget al,1992). But the research formasonry walls supported on frame is seldom. Therefor in this paper thetest results willbe gavenand try to calculate the bearing capality using plastic limitanalyses methods for the perforated brick masonry walls supported on frame.

EXPERIMENTAL RESEARCH

Test Program

Theinvestigation' s objective are tostudy behaviour, modesoffailare andeffects of several important factors on modes of failure andultimate strength forthe wall-beams supported frame. Six specimens onthe perforated brick masonry wallssupported on single-storey and onespan frame weretested to failure in the laboratory. The height-span ratio $h_w/l_o=0.485, 0.628$, beam-column rigid ratio $K_{mb}/K_{mc}=0.5, 0.605, 1.185$ and percentages of bottom longitudinal bars of beams $\rho=A_s/bh_o=0.56\%, 0.80\%, 1.2\%$. The compressive strength of concretecube f_{cu}, the tension strength of steel bars fy, the compressive strength of perforated brick, mortarand masonry f_1, f_2, f_m andother factors of specimens are shown in Table 1. Each specimenis loaded directlyuniformlyon thetopface bymeans of twosymmetric hydraulics and loadingcellsthrough the doublelayer distributingbeams and one layer steel slabs. Inthetests the midspan deflection of beam, the longitudinal reinforcementstrainsonmidspanandsupportsof beam, the vertical reinforcement strains on top and bottom of column, the horizontalstrains on midspanvertical section of wall, the vertical and horizontal strains on interface between wall and beam, cracking loads and failue load, etc are measured. The typical testing arrangement is shown in Fig.1.

Behaviour Under Loads

Testresultsindicatethatthewalls-framecompositestructuresgothrougth elastic, unelastic workand failure stages. When the loads are less than 0.35 F_u (F_u isultimate loads), the walls-frameis in elastic stage. The midspan deflection of beam the strains in steel bars and concrete of beam and column are verysmalland varie linearly with the increaseof theloads. Thestrainsdistribution in top and bottom section of column accordingtoBernonlli's theorem, butthe strains distribution in midspan and supports section of wall-beamdonot. Whenthe loads increase to about 0.35 F_u, the first vertical flexural crack will appear on the midspan of beam. After that other vertical cracks will appearsuccessively anddevelop to walls. But variation of stress and deformationis unremarkable. Whenthe loadsincrease to about 0.85 F_u, with appearingand developing of diagonal cracks in walls or beam near supports, the behaviourofwalls-frame changesa lot, i.e thebeam-action weakens andthe arch-action strengthens in walls-beam. The walls-framewill goto failure stage and a system for transferring

Table 1——Test data of specimens

specimens	FW 1	FW 2	FW 3	FW 4	FW 5	FW 6
dimension of specimens (mm)						
beam $b_b \times h_b$	120×120	120×160	120×120	120×120	120×120	120×120
Column $b_c \times h_c$	120×120	120×120	120×100	120×120	120×120	120×100
Wall $h_w \times h$	970×112			1130×112		
span l_o	2000			1800		
h_w / l_o	0.485			0.628		
Column Hcn	940	910	640	840	840	570
K_{mb} / K_{mc}	0.5	1.185	0.605	0.5	0.5	0.605
reinforcement of beamsand columns						
top of beam	2Φ6					
bottom of beam	2Φ8	2Φ10	2Φ8	2Φ8	2Φ10	2Φ6
ρ (%)	0.84	0.98	0.84	0.84	1.29	0.56
column	4Φ6					
steel hoop	Φ6@80					
strength of material (N/mm^2)						
concrete f_{cu}	34.09	31.38	31.87	33.71	28.93	26.28
brick f_1	22.41			16.49		
mortar f_2	9.77	7.44	9.0	16.33	14.06	16.92
masonry f_m	4.90	5.10	5.12	3.39	4.34	5.04
strength of steel bars (N/mm_2)						
diameter	Φ6	Φ8	Φ10	Φ6	Φ8	Φ10
yield strength f_y	424.3	327.4	309.0	420.0	320.1	285.6
ultimate strength f_{su}	456.1	389.7	394.7	451.9	447.3	373.1
percentage of elongation (%)	20.0	31.7	16.0	19.2	25.9	37.7

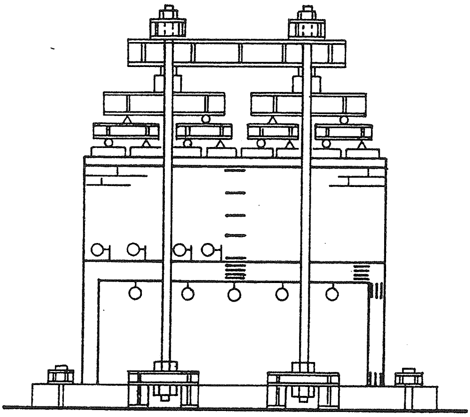

Fig.1 Testing arrangement

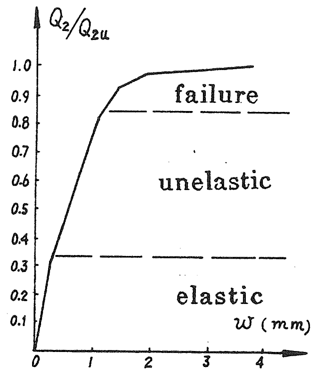

Fig.2 Load-deflection curve

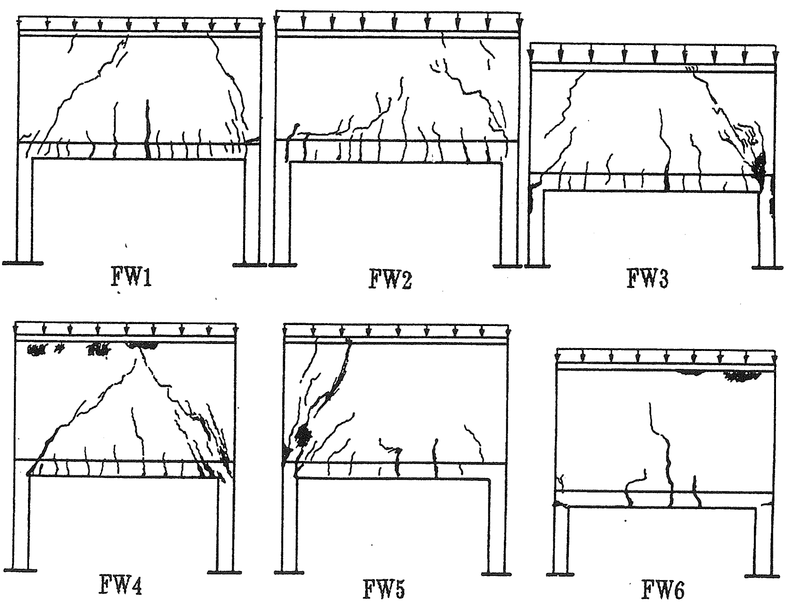

Fig.3 Failure crack patterns

forces like arch and frame composite structures is formed finally. The testresults are shown in Table 2.

Modes of Failure

The following failure modes will appear possiblely for walls-frame.

Flexural Failure. It is the failure caused by yielding of longitudinal bars in beam or column. When the height-span ratio h_w/l_o of walls and the percentageof longitudinal bars ρ of beam are small. At first a tensile-flexure plastic hinge is formed in midspansection of beam. Because the second plastic hinge appears in supported section of beam or topsectionofcolumn, two deferentfailure mechanismsare formed. It is the failure mechanismsaroused by yieldingof vertical bars near outside of column, i.e. thatthe compressive-flexureplastichingeis formedand caused by yielding of longitudinal bars of top of beam near supports, i.e. that the flexural plastic hinge is formed.

Table 2—Test results and comparison

specimens	FW 1	FW 2	FW 3	FW 4	FW 5	FW 6
characteri-stic loads and modeof failure (kN/m)						
vertical cracking on beam	47.0	56.3	19.0	72.5	82.9	63.2
inclined craking on beam	94.0	122.1	123.2	196.9	176.2	/
yielding of bars in beamon midspan	94.0	/	123.2	/	/	137.1
yielding of bars in beamon supporte	112.8	/	131.5	/	/	/
yielding of bars in column on top	/	/	/	/	/	170.8
inclined cracking on walls	104.3	122.1	123.2	196.9	176.2	/
local crushing	/	/	146.9	196.9	178.2	/
ultimate bearing capacity	112.8	131.5	146.9	217.6	178.2	171.3
modeof failure	F-S	S(T)	F-S(L)	S(C)(L)	S(C)(L)	F
comparison with calculated valwes						
(test values) Q_2^t(kN/m)	112.8	/	131.5	/	/	170.8
values by [4] or [5] Q_2^c(kN/m)	115.4	/	123.9	/	/	155.2
Q_2^c/Q_2^t	1.023	/	0.942	/	/	0.909
test values V^t (kN)	106.5	124.1	140.5	170.1	150.5	/
values by [6] and [2t] V^c(kN)	116.4	141.2	122.5	153.7	158.0	/
V^c/V^t	1.093	1.138	0.872	0.904	1.050	/

Notes: F—flexure failure; S—shear failure; T—diagonal tensile failure;
L—local crushing; C—diagonal compressive failure

Shear Failure It is failure aroused due to compressing or tensioning inclinedly of masonry walls near supports and crushing or splitting inclinedly of concrete of beam-column joints. The longitudinal bars of beam and the vertical bars of column are not yielded. There are two shear failure modes. The diagonal tension failure is failure aroused due to principal tensile stress σ_{tp} is more than tension strength f_{mt} of masonry walls and when h_w/l_o is minor usually. The diagonal cracks are developed by toothing mortar joint tabularly, and the angle of inclination of the diagonal cracks is less than 45°. The diagonal compressive failure is failure aroused due to principal compressive stress σ_{cp} more than compressive strength f_m of masonry walls and when h_w/l_o is major usually. The diagonal cracks are developed througth the brick and level mortar joint steaply. Many short parallel inclined cracks appear and soon the masonry between placed points of loading (about $^1/_3$~$^1/_4$ span from supports) and column is crushed inclinedly. The angle of inclination of diagonal cracks is about 55~60°.

Flexure-shear Failure It is the failure which occures due to yieding of the bottom longitudinal bars in midspan section of beam and crushing inclinedly of the masonry walls between placed points of loading and column. After that the top longitudinal bars in supports section of beam or the vertical bars in upside section of column are yielded and the flexural failure mechanisms are formed.

Local Crushing Failure It is the failure which occures due to the stress concentraction is very large in masonry and concrete above column near supports and the local stress more than local compressive strength. When h_w/l_o is larger, the local compressive failure of masonry near the supports and the local crushing of concrete above column appear easily. The failure crack patterns of all specimens are shown in Fig .3.

PLASTIC LIMIT ANALYSES

Basic Assumptims and Failure Mechanism

The steel bars is a rigid-plastic material. Its stress-strain curve is shown in Fig.4. Assume $f_y=|f_y'|$.

The masonry is also assumed to a rigid-plastic material. Its stress-strain curve is shown in Fig.5. Its yielaling strength at uniaxial stress is:

$$f_{mp}=\gamma_m f_m \qquad [1]$$

Where, γ_m is plastic effective compressive coefficient,

$$\gamma_m=0.75-f_m/10 \qquad [2]$$

The yielding condition at plane compressive stress is shown in Fig.6.

$$\sigma_1=f_m \qquad (\text{when } 0\leqslant\sigma_2\leqslant f_m)$$

$$\sigma_2=f_m \qquad (\text{when } 0\leqslant\sigma_1\leqslant f_m) \qquad [3]$$

Ignore the act of concrete and masonry at tension.
Assume that the frame-beam flexural mechanism, the frame-beam and column flexural mechanism and walls-beam shear mechanism are occured possiblely accordmg tothe test results. It is shown in Fig.7.

Flexural Capacity

The frame-Beam Flexural Machaniam We may obtain the upperbound solution according to the plastic virtural workeguation. The enterforce work and interforce workare, repectively,

$$W_e = Q_2 l_n^2 \theta / 4 \qquad [a]$$

$$W_i = 2(M_{b2} + N_{b2} z + M_{be2}) \theta \qquad [b]$$

$W_e = W_i$, then

$$Q_2 = 8(M_{b2} + N_{b2} z + M_{be2}) / l_n^2 \qquad [4]$$

The Frame-Beam and column Flexural Mechanism We may obtain also the upperbound solution. The exterforce workis calculated by formula [a]. Theinterforce work is:

$$W_i = 2(M_{b2} + N_{b2} z + M_{cu2} + N_{cu2} e') \theta \qquad [c]$$

$W_e = W_i$, then,

$$Q_2 = 8(M_{b2} + N_{b2} z + M_{cu2} + M_{cu2} e') / l_n^2 \qquad [5]$$

where,
Q_2——uniform loads on topface of walls-frame,
l_n——clear span, face to face of column,
l_o——computation span take $l_o = l_c$, l_c is span centre to centre ofcolumn,
M_{b2}——moment in midspan section of beam,
N_{b2}——axialtension force in midspan section of beam,
M_{be}——moment in supported section of beam,
M_{cu}——moment in upside section of column,
N_{cu}——axialcompressive force of column,
z——internal moment arm,
e′——distance between axial force and interside bars of column.

Shear Capacity

Lower Bound Solution. We may divide a walls-beam into 11 parts (Fig.8.). Thestress of part 2 and 6 egual 0.1 and 5 are uniaxial compressive parts. 3 and4 are bisxial compessive parts. 7 is uniaxial tension part. Assume part 1 to achieve the yielding strength of masonry, then the shear of vertical section is shear correspouded with static possiple stress field. We may obtain the lower boundsolution for theshear strength in the walls of walls-frame.

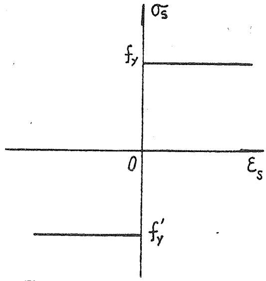

Fig.4 Stress-strain curve of steel bars

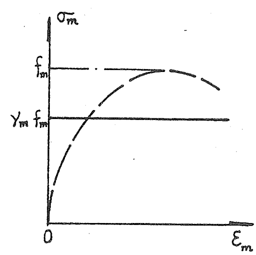

Fig.5 Stress-strain curve of masonry

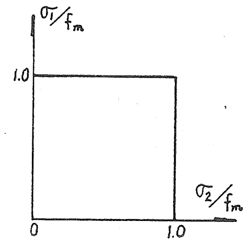

Fig.6 Yielding condition at plane stress of masonry

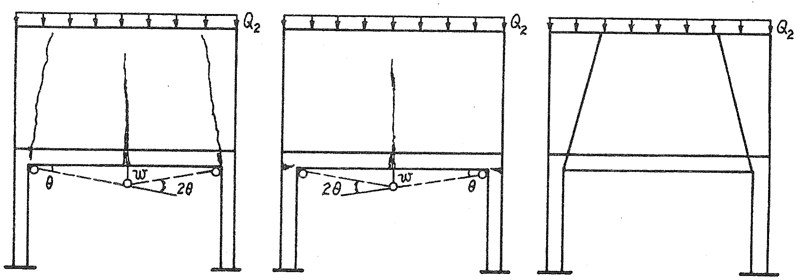

Fig.7 Possible mechanismes of walls-frame

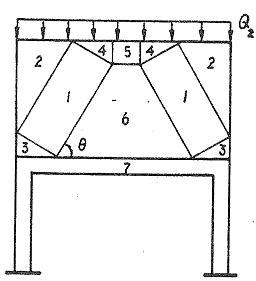

Fig.8 Divided parts of walls-frame

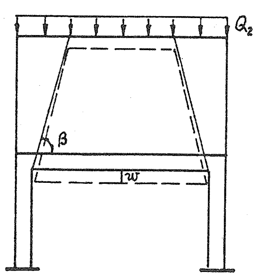

Fig.9 Shear mechanism of walls-frame

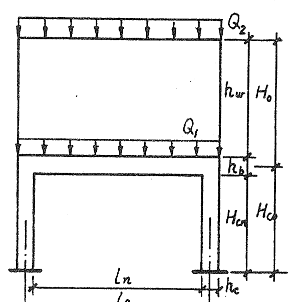

Fig.10 Calculated simplified diagram

$$V_m = \gamma_m f_m h h_w \sin 2\theta \qquad [d]$$

Solve the maximum, $dV_m / d\theta = 0$, then $\theta = 45°$,Obtain

$$V_m = \gamma_m f_m h h_w / 2 \qquad [6]$$

UpperBoundSolution. Assume that the possible failure mechanism is shawn in Fig.9. We may obtain alsothe upper bound solution according to virtural work eguation. The exterforce work is

$$W_e = Q_2 l_o w = 2(V_m + V_b) w \qquad [e]$$

Where V_b is the shear subjected by beam. We discusse only the shear V_m subjcted by the masonry walls. Then,

$$W_{me} = 2V_m w \qquad [f]$$

The interforce work is

$$W_{mi} = \gamma_m f_m h h_w (1-\cos\beta) w / \sin\beta \qquad [g]$$

$W_{me} = W_{mi}$, then,

$$W_m = (1/2)\gamma_m f_m h h_w (1-\cos\beta) / \sin\beta \qquad [h]$$

Solve the minimum, $dV_m / d\beta = 0$.then $\beta = 90°$ .We obtain the upper bound solution is indentical with thelower bonud solution. Therefor the formula [6] is real shear capacity of the masonry wallsofwalls-frame.

Comparison withTest Results
Thecomparison of calculated values according to formula [4] with test results of specimensFW1,FW3andcalculated values accordingto formula [5] with test result of specimenFW6is shown in Table2 andindicates: the averageratio of calculated valueto test data $\mu = 0.958$,coefficient of variation $\delta = 0.061$. Thecomparison of calculated values according to formula [6] with test results of specimens, in which are occured the shearfailure ortheflexural-shear failure, is also shown in Table 2 and indicate: numbers ofspecimens $n=5$, averageratio of calculated value to testdata $\mu = 1.011$, coefficient ofvariation $\delta = 0.116$, the theoritcal values according totheplasticlimitanalysestally with the test results.

SUGGESTIONS OF DESIGN FOR WALLS ON FRAME

General Specify
Thespan of walls-frameis not larger than 9m generally. Total heightof wallsabove framedo not exceed 15m usually and mustbe not less than $0.4\ l_o$. The height of beam is not less than $l_o/12$. Theothers arethe same as general frame and simple wall-beams.

Loads on Walls and Frame

Theloads on the walls-frame havetwo parts. Theloads on topface of beam Q_1include thedead weight of beam and the dead andliveloads of the firstfloor slabs. Theloads on top face of walls of walls -frameQ_2 are calculated by the following formulas.

$$Q_2 = g_w + \Psi Q_i \qquad [7]$$

$$\Psi = 1/[1+2.5 b_f h_f/(l_o h)] \qquad [8]$$

where,

g_w——dead weight of all walls above frame,

Q_i——deadand live loads of all floor slabs, excepting the first floor slab,

b_f, h_f——width and depth of the plange walls, respectively.

Thecalculated simplified diagram of the walls-frame is shown inFig.10.

Culculation for Interforce

The interforce of frame under uniformloads Q_1 may be calculated according togenaral structural analyses. M_{b1}, M_{be1}, V_{b1} are midspan. moment, end moment and end shear of beam, respectively. M_{cu1}, N_{cu1}, M_{cl1}, N_{cl1} aremoment,axial compression of upper endand moment, axial compressionof lowerand of columns,respectively.The interforce of the walls-frameunderuniform loadsQ_2 may be calculated by the following formulas. For the columns of frame, have:

$$M_{cu2} = Q_2 l_o^2/90 \qquad [9]$$

$$M_{cl2} = Q_2 l_o^2/180 \qquad [10]$$

$$N_{cu2} = N_{cl2} = -Q_2 l_o/2 \qquad [11]$$

For the beam of frame, have:

$$N_{b2} = 0.188 Q l_o \qquad [12]$$

$$N_{be2} = 0.094 Q l_o \qquad [13]$$

$$M_{b2} + N_{b2} z + M_{be2} = Q_2 l_o^2/8 \qquad [14]$$

$$M_{b2} + N_{b2} z + M_{cu2} + N_{cu2} e' = Q_2 l_o^2/8 \qquad [15]$$

$$V_{b2} = (p_{omax}/S)(S - h_c/2)^2/2 \qquad [16]$$

$$S = 0.812\ h_b \sqrt[3]{(1+2k_{mc}/k_{mb}) E_s/E_u} \qquad [17]$$

$$p_{omax} = Q_2 l_o/S \qquad [18]$$

$$z = 0.1(5.0 + l_o/H_o) H_o \qquad [19]$$

Take, $1.0 \leqslant l_o / H_o \leqslant 2.5$.
For the walls of the wall-frame, have,

$$V_m = Q_2 l_n / 2 - V_{b2} \qquad [20]$$

Calculation for Bearing Capacity
The bearing capacity of column may be calculated according to the reinforced concrete eccentric compressionmembers to use the following interforce. For the upper end section, have,

$$M_{cu} = M_{cu1} + M_{cu2}, \quad N_{cu} = N_{cu1} + N_{cu2} \qquad [21]$$

For the lower end section, have,

$$M_{cl} = M_{cl1} + M_{cl2}, \quad N_{cl} = N_{cl1} + N_{cl2} \qquad [22]$$

The bearing capacity of beam maybe calculated according to the reinforced concnete eccentric tension members to use the following interforce. For the midspan section, have,

$$M_b = M_{b1} + M_{b2}, \quad N_b = N_{b2} \qquad [23]$$

For the end section, have,

$$M_{be} = M_{be1} + M_{be2}, \quad N_{be} = N_{be2} \qquad [24]$$

The shear capacity of beam may be calculated according to the reinforced concrete eccentric tension members to use the following formulas.

$$V_b = V_{b1} + V_{b2}, \quad N_{be} = N_{be2} \qquad [25]$$

$$V_b = V_{cs} = 0.07 f_c b h_o + 1.5 f_y h_o (A_{sv} / s) - 0.2 N_{be} \qquad [26]$$

The shear capacity of walls of walls-frame may becalculated by the following formula.

$$V_m \leqslant 0.5 \gamma_m f_m h h_w \qquad [27]$$

The local compressive capacity of the masonry walls above beam on supports may be calculated according to the following formulas.

$$Q_2 \leqslant \zeta h f \qquad [28]$$

$$\zeta = 0.25 + 0.08 h_f / h \qquad [29]$$

Detailing

The strength grade of concrete, brick and mortar must be not under C20, MU10 and M5, respectively. The width and height of opening in walls must be not larger than 0.3 l_o and 5 $H_w/6$, respectively. The depth of walls and beams must be not less than 240mm. The minimum section size of column is 240×240mm. The minimum pecentages of longitudinal bars in the beams and columns are 0.5% and 0.4%, respectively. The longitudinal bars in beams and columns must be reliably anchored and the usable design anchorage lengthes of longitudinal bars for the walls-frame must be incresed 5d than for the general frame.

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