Published by Elsevier Science Ltd 1996
Paper No. 564. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

SEISMIC RESPONSE MULTI-LAYER BEARING WALLS

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ABSTRACT

This paper presents part of a complex study of buildings with 3-layer bearing walls designed for seismic regions. The 3 layer (sandwich) walls consist of an inner layer of reinforced concrete, or concrete without reinforcement, and two external layers made of hollow brick masonry, or limestone masonry, or any other available local material. The inner layer is the main bearing elements (bearing vertical as well as lateral seismic loads). It is the main but not the only bearing element. The investigations show that the 2 external layers are carrying a part of the vertical as well as horizontal load. Important is that during a strong earthquake, due to the slips between blocks in the external layers, dry friction can occur causing energy dissipation which is useful, decreasing seismic accelerations and displacements. It sums up the results of layer strength and layer interaction tests under lateral loading in test pieces and actual 3-layer bearing walls. The necessary design parameters used for defining the bearing capacity of walls made of combined materials were received in the course of tests. The paper also gives a based on test-obtained deformation diagrams computer analysis of 3-layer bearing walls buildings response to actual earthquake accelerogram.

KEYWORDS

3-layer bearing walls, limestone masonry, concrete or ceramic blocks, inner and external layers.

INTRODUCTION

In Leninakan, Armenia, more than 90% of frame buildings collapsed, more than 25000 people were killed during the Armenian-Spitak earthquake of December 7, 1988, and most of them in frame buildings (*J.M.Eisenberg 1991*). During the Erzincan-Turkey earthquake of March 13, 1992, also the reinforced concrete frame building demonstrated poor seismic behaviour (*R.Yarar et. al., 1993*). Around 80% of frame buildings collapsed or were heavily damaged. But not only frame buildings have heavy damages. During Erzincan earthquake, 1992, in the Uzumlu city many buildings with hollow brick walls damaged or collapsed. Much better was the behaviour of reinforced concrete prefabricated large panel buildings during the Spitak, 1988, earthquake, as well as during many other recent destructive earthquakes. No one of 16 nine-story large panel buildings in Leninakan have sufficient damages during the Armenia-Spitak, 1988, earthquake. Nobody inside these buildings was even injured.

In many seismic acetic areas in Russia, Turkey as well as in other countries - hollow bricks limestone, ceramic blocks and other local materials are popular, are highly distributed and are often used in bearing walls masonry. But these materials being brittle and having low strength are not adequate as bearing wall material in seismic dangerous areas. Monolithic or large panel prefabricated walls being save enough in some cases have some shortcomings.

In many cases the good solution is to use combined RC-stone masonry walls, or RC-hollow brick walls or some others materials of this kind. The recent destructive earthquakes in Russia, in Turkey, in Greece, have given a new pulse for using 3-layer RC-stone masonry (sandwich) wall buildings in earthquake prone areas.

In the early 80-th in Kishinev, Moldavia, have been built several 9 to 12 story building with 3-layer walls. The external two layers were of limestone masonry and the inner layer of RC. The seismic behaviour of these buildings was excellent during the 1986 Karpat earthquake. They have even less visual cracks then large-panel RC wall buildings or Monolithic bearing wall buildings (*J.M.Eisenberg* 1993).

This paper is a small contribution to the subject of seismic behaviour of 3 LWB. The object of the paper is to describe some results the influence of the external layers on the earthquake behaviour of the sandwich walls:
* the layers interaction and which of the total strength of the wall under simultaneous action of dead loads and other vertical loads combined with seismic loads;
* the experimental hysteretic curves and their evolution during seismic loads;
* computer analysis inelastic systems with different parameters of the external layers, with different high stories;
* estimation of ductility factor for 3-layered wall building.

Some studies were carried out in ITU (Istanbul), in Kishinev University (Moldavia), in Moscow Research Institute of Building Structures.

CHARACTERISATION 3-LAYER (SANDWICH) WALL BUILDING

The 3-layer walls are walls which consist of inner layer of reinforced concrete, or concrete without reinforcement, and two external layers made of hollow brick masonry, or limestone masonry, or concrete or ceramic hollow blocks, particularly, Portugal "Interblock" dry masonry (without mortar) or light concrete without hollow blocks, or adobe, or any other available local material (Fig. 1).

The inner layer is the main bearing elements (bearing vertical as well as lateral seismic loads). It is the main but not the only bearing element. The investigations show that the 2 external layers are carrying a part of the vertical as well as horizontal loads. Important is that during a strong earthquake, due to the slips between block in the external layers, dry friction can occur causing energy dissipation which is useful, decreasing seismic accelerations and displacements. The external layers are formworks during concreting the inner layer.

WALL PIECES TESTING

Test aims. 15 wall pieces with different combination of materials in inner and external layers were tested as to central and excentral compression and to simultaneous vertical and lateral loads action under direct and alternate loading. Tests were carried out in order to find out what major factors determine the specific behaviour of 3-layer bearing wall with cast-in-situ reinforced concrete inner layer when simulating earthquake induced vertical and lateral forces. The results of the piece testing were necessary for defining some of the calculated parameters used for theoretical analysis of the bearing capacity of 3-layer walls made of combined materials.

Test pieces were divided into two groups in accordance with the construction design and the number of layers. Group I comprised one layer pieces (blocks, masonry, cast-in-situ concrete). Group 2

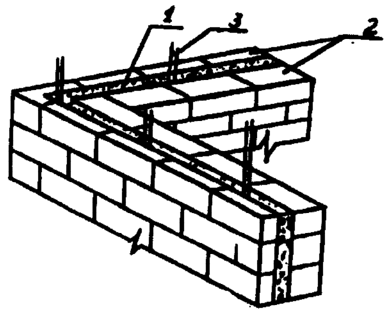

Fig.1. 3-layered wall building
1 - inner layer of reinforsed concrete; 2 - two external layers of brick masonry; 3 - reinforsing case

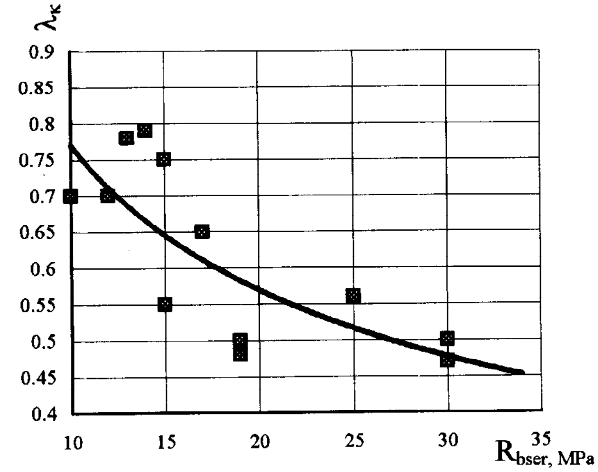

Fig. 2. Experimental correlation between interaction factor λ_k and the inner layer concrete relative strenght

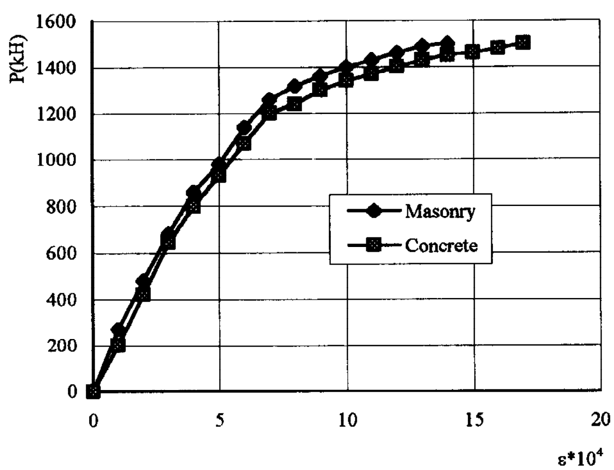

Fig. 3. Gorisontal forse displasement diagram

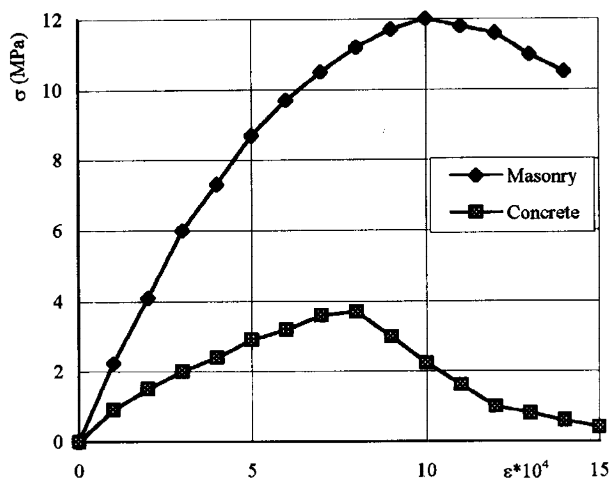

Fig. 4. Stress relation to deformation for masonry and concrete within 3-layer section

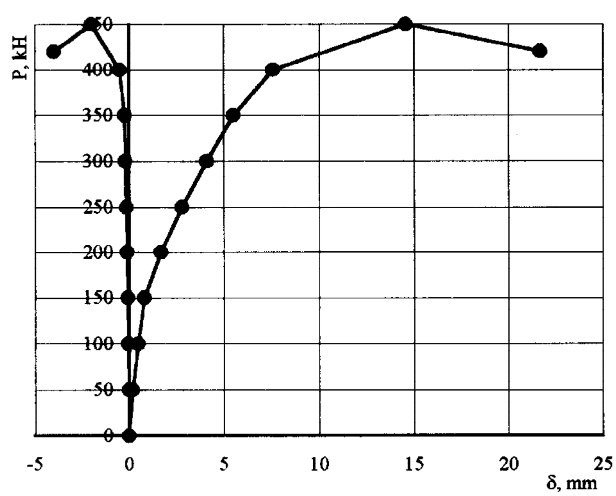

Fig 5. Diagram of the test pattern main area diagonal deformations

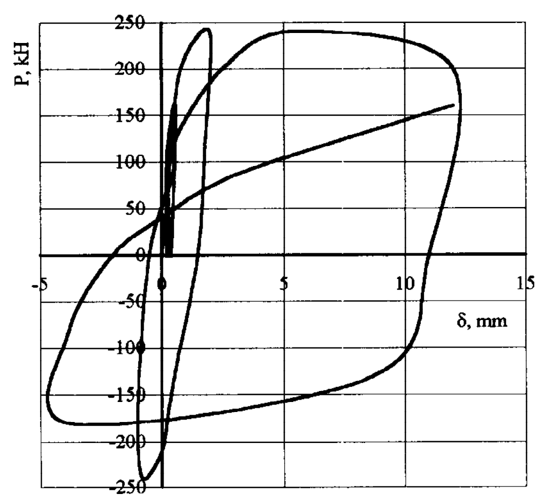

Fig. 6. Horisontal forse-displasement diagram for 3-layer wall

included 3-layer pieces (cast-in-situ or reinforced concrete inner layer and two external stone material layers. The pieces were tested as to static vertical and lateral loads with the gradual load increase until intensive deformation developed leading to the piece's collapse.

Materials and models. Test wall pieces were made of: 1)small sawn limestone blocks (390 x 190 x 188 mm); 2) brand 75 hollow brick (130 x 190 x 185 mm); 3) brand 75 solid red brick (250 x 120 x 65 mm); 4) brand B10 common concrete for inner layer 120 and 220 mm thick; 5) brand B7.5 ceramsite concrete 120 mm thick. Continuous wall pieces sized: a) 1200x1300x400(500, 600) mm; b) 1500 x 900 x 360 (120, 190) mm were used for tests.

Central compression tests

Two groups of wall pieces were used for central compression tests:
* test patterns of masonry without transversal bond;
* test patterns of 3-layer walls having 120 and 220 mm thick concrete layer.

Masonry test pattern wall pieces. Test data analysis made it possible to find empiric coefficients for masonry materials which are part of the masonry ultimate strength under compression formula:

$$R_u = AR_{u1}(1 - a/(b + R_{u2}/2R_{u1}))\eta \quad (1)$$

$$A = 10 + R_{u1} / (10m + n\, R_{u1}) \quad (2)$$

where R_{u1}, R_{u2} - stone and mortar ultimate strength under compression; a, b - empiric coefficients, depending on masonry type; η - corrector factor for low brand mortar masonry (when $R_{u2} \geq 0.4$ MPa $\eta = 1$; when $R_{u2} = 0$ $\eta = 0.75$); A - constructional coefficient depending on stone strength.

3-layer test pattern wall pieces. Figure 2 represents their test results. One of the test aims was to measure coefficient of masonry utilization in complex structures which is determined according to the formula

$$\lambda_k = (P_u - m\varphi_b A_b R_{b,ser}) / \varphi_k A_k R_u \quad (3)$$

where P_u - bearing capacity of complex structure; A_k, A_b - cross-sectional area of the wall masonry and concrete layers; m = 0.9 - coefficient considering concrete heterogeneity; R_u - masonry ultimate strength under compression according to formula (1); $R_{b,\ ser}$ - concrete prism strength; φ_k, φ_b - coefficient considering masonry and concrete layer flexible.

Figure 2 shows that λ_k values are within 0.45-0.79 limits. It is noteworthy that λ_k value goes down when concrete strength increases. The curve from the test results permits to evaluate λ_k under different concrete prism strength values (concrete brand). High λ_k values indicate active role of masonry in forming 3-layer wall bearing capacity.

Figure 3 represents relation of test wall pieces deformation greatly exceed masonry ultimate compressibility ($\varepsilon_{max,\ cr}$). As masonry layer and concrete layer deformations are practically equal at the time of collapse, it may be assumed that masonry stresses have passed by this time the maximum equal masonry ultimate strength and are below it. Fig. 4 depicts masonry and concrete layers dynamics. Knowing the concrete ultimate compressibility and $\sigma = f(\varepsilon)$ relation for masonry it is possible to calculate stresses in masonry ($\sigma_{ms} < R_{ms}$) at the moment of complex structure bearing capacity exhaust. Then λ_k coefficient can be assumed as σ_{ms} / R_{ms} relation.

Masonry-cast-in-situ wall piece testing as to vertical and lateral loads

A large-sized 3-layer wall piece was made for the test. During the test all deformations, loads and appearing cracks were being fixed. Under ultimate loading the compressed area collapsed, the stretched out outline reinforcement bars lost fluidity state and reached self-strengthening zone. Deformation diagrams in Fig. 5 show that the test wall piece's extended diagonal was deformed much greater than the compressed diagonal. The structure collapse on the whole was rather "mild". Plasticity coefficient reached 3.9 value.

Masonry-cast-in-situ wall piece testing as to alternate loads

Masonry-cast-in-situ test wall piece sized 1.5 x 0.9 m(h) comprised 3 layers: two external 12 cm thick masonry layers and the inner 12 cm thick ceramsite concrete layer. The test wall piece was cyclically subjected a to constant vertical loading with variable eccentricity longside cyclically changed lateral alternate loading. The charts in Fig. 6 represent the test piece top horizontal displacement under cyclic loading. They show a rather high test pieces deformation plasticity. Plasticity coefficient $\mu = \delta_{max} / \delta_y$: for the test piece upper part $\mu = 21 / 0.7 = 30$; along diagonal deformations $\mu = 12.4 / 2.4 \approx 5$, which is higher than for ordinary masonry.

Conclusions

1. In the result of masonry patterns testing empiric coefficients values for calculating ultimate strength for masonry of local materials have been established. Coefficients λ_k of masonry utilization in complex structures for concrete of different strength have also been obtained. 2. It has been found out that provided the design and construction technology are correct, external masonry layers and inner concrete layer "work together" until structure collapses and structural strength can be enhanced 2-4 times due to masonry-concrete bond. 3. 3-layer wall piece tests showed plastic and not brittle structural collapse. This fact is very important from the point of view of the structure's seismic behaviour.

COMPUTER MODELLING

The purpose of computer modelling is to analyse 3-layer bearing wall structure seismic response to actual earthquake accelerograms. To reach this purpose the following has been done: calculated earthquake accelerograms have been chosen; characteristics of a design dynamic model of a building with 3-layer bearing walls have been described; free and forced oscillation behaviour has been calculated.

Design dynamic model of a 3-layer bearing wall structure

Frame rod system with lumped of masses at each storey level has been assumed to be design structure scheme. Bending and shift deformations for column diaphragms have been taken into account. The fixation at the support points is calculated absolutely rigid. Seismic forces act at joints synchronically, wave phenomena not being considered. "Force-displacement" characteristics have been simulated by the actual dependence received in the course of experimental, stiffness and strength degradation not taken into consideration. Structure elements axle stiffness has been assumed to be constant and "P-δ" effect has not been considered. It has been supposed that individual wall bearing layers have similar displacements.

Calculating of rigidity characteristics

3-layer bearing wall structure R' rigidity matrix has been built on the basis of displacement method. Constructive element - wall partition - is considered to be the finite element. Reactive moment and lateral force at each element's joint are equal to

$$M = 6EI/(l^2(1+k)); \qquad Q = 12\,EI/(l^3(1+k)); \qquad k = 12\,\mu\,EI/(l^2GA) \tag{4}$$

where E - modulus of elasticity; I - section inertia moment; A - section area; G - shear modulus; l - element's length; μ - coefficient depending on section form. All structural elements' R' rigidity matrix has been received by summing up R'_m rigidity matrix components of all system elements according to the formula $R' = \sum R'_m$ (m changes from 1 to n, n - elements number). Structural elements strength characteristics have been taken from the experiment.

Algorithm of solving equation of the structure motion at elasto-plastic stage

Differential equation of incremental motion have been used for the analysis

$$M\,\Delta\ddot{Y} + B\,\Delta\dot{Y} + R'\Delta Y = \Delta P \tag{5}$$

where ΔY, $\Delta\dot{Y}$, $\Delta\ddot{Y}$ - incremental displacement, velocity and acceleration vectors accordingly; M - mass matrix; B - damping matrix; $R' = \partial R / \partial Y$ - instant or tangent matrix of system rigidity at motion moment t

$$R' = \begin{vmatrix} \partial R_1/\partial Y_1 & \ldots\ldots\ldots\ldots & \partial R_1/\partial Y_n \\ \ldots\ldots\ldots & \ldots\ldots\ldots & \ldots\ldots\ldots \\ \partial R_n/\partial Y_1 & \ldots\ldots\ldots\ldots & \partial R_n/\partial Y_n \end{vmatrix} \tag{6}$$

Wilson numerical method has been used for solving differential equation, with acceleration at integration step changing linearly. In making up $R'M_1B$ - matrices for bearing structure the systematic approach has been applied. Finite matrices have been formed from separate elements matrices in accordance with the system topology. In order to consider inner and external resistance matrix B has been assumed as a sum

$$B = \beta_1 R' + \beta_2 M \tag{7}$$

β_1, β_2 - coefficients have been chosen on the condition that oscillation decrements which correspond to damping free oscillations made having two characteristic forms of eigen oscillation are frequency independent.

Results of calculation and conclusions

The following variable parameters have been taken for the analysis:
* inner layer - reinforced concrete 100 , 120 , 140 , 160 and 180 mm thick;
* external layer - sawn limestone 190 mm thick; masonry 250 and 120 mm thick;
* number of a building's storeys - 3, 5, 7 and 9;
* design accelerograms have been assumed to be Bucharest, NS, 04.03.1977; Loma-Prieta, EW, 17.10.1989; El-Centro, NS, 17.05.1940.

Having computed free oscillation behaviour, dynamic model frequencies and forms have been obtained for 3-layer bearing walls with different combinations of materials used. Oscillation periods within 0.15 - 0.25 sec limits indicate considerable stiffness for stone-cast-in-situ structures. Eigen oscillation forms are of bending-shifting character.

The following data have been printed as a result of solving equations of motion: displacement, velocity and acceleration oscillograms; system reaction; displacement, velocity and acceleration maximum values. Figure 7-9 shows displacements corresponding to maximum seismic loads. Diagrams in Fig. 10-12 display maximum ratio of R_{max} reaction to system R_y ultimate elasticity reaction for each storey. The following conclusions have been drawn from the carried out analysis:

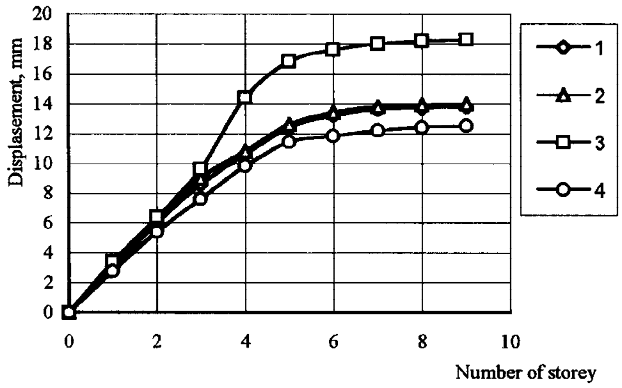

Fig.7. Maximum displacement (El-Centro);
1- monolitic walls; 2- limestone masonry and concrete within 3-layer section; 3- brick masonry and concrete within 3-layer section (d=120 mm); 4- the same as 3 but d= 250 mm

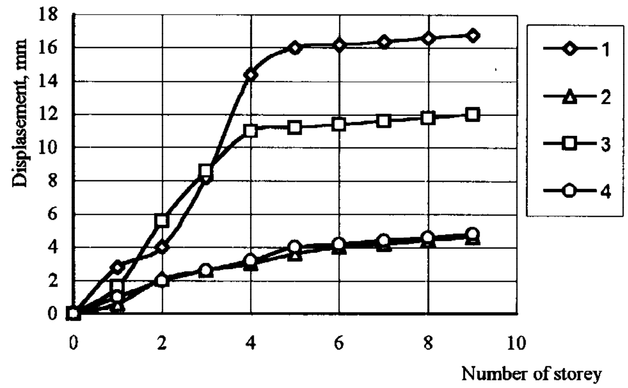

Fig. 8. Maximum displacement (Loma-Prieta);
1- monolitic walls; 2- limestone masonry and concrete within 3-layer section; 3- brick masonry and concrete within 3-layer section (d=120 mm); 4- the same as 3 but d= 250 mm

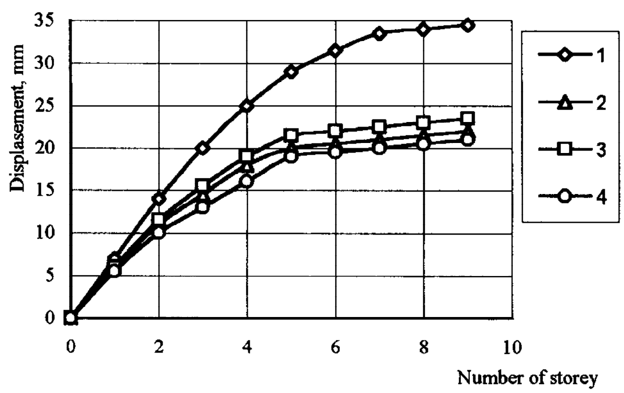

Fig. 9. Maximum displacement (Bucharest);
1- monolitic walls; 2- limestone masonry and concrete within 3-layer section; 3- brick masonry and concrete within 3-layer section (d=120 mm); 4- the same as 3 but d= 250 mm

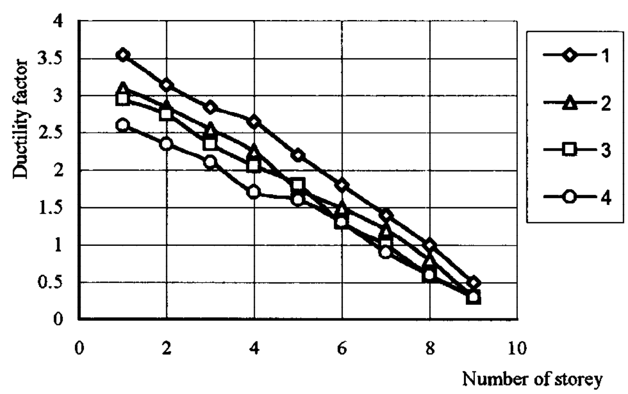

Fig. 10. Maximum ductility coefficient (Bucharest);
1- monolitic walls; 2- limestone masonry and concrete within 3-layer section; 3- brick masonry and concrete within 3-layer section (d=120 mm); 4- the same as 3 but d= 250 mm

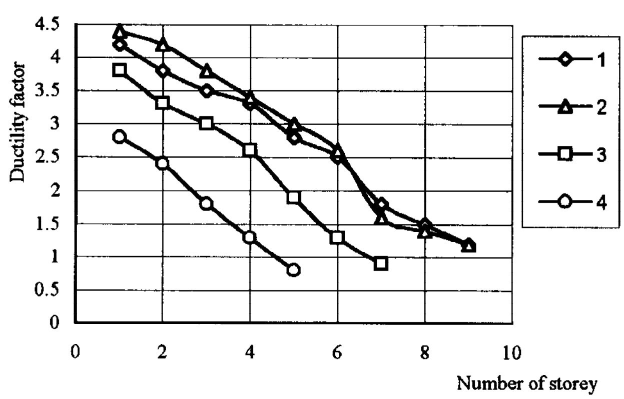

Fig. 11. Maximum ductility coefficient (El-Centro);
1- monolitic walls; 2- limestone masonry and concrete within 3-layer section; 3- brick masonry and concrete within 3-layer section (d=120 mm); 4- the same as 3 but d= 250 mm

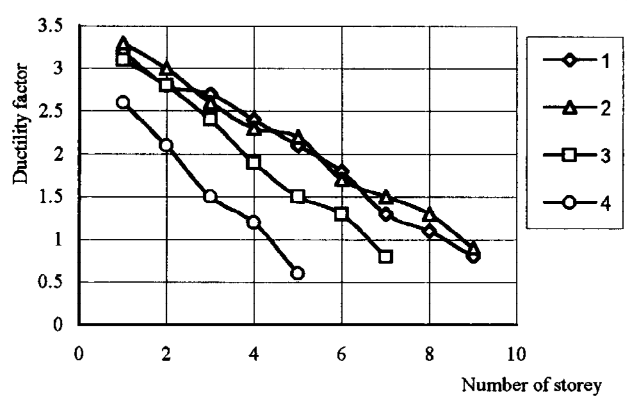

Fig. 12. Maximum ductiliti coefficient (El-Centro);
1- monolitic walls; 2- limestone masonry and concrete within 3-layer section; 3- brick masonry and concrete within 3-layer section (d=120 mm); 4- the same as 3 but d= 250 mm

* when subjected to a force, the wall with external masonry layers 250 mm thick has the smallest displacement as compared to cast-in-situ concrete or other walls;
* the relative displacements compared, it has been found out that relative displacements in 3-5 storey buildings are smaller than in 7-9 storey ones; when the number of storeys is reduced from 9 to 5 plastic deformation level decreases by 1.5;
* practically all storeys are deformed beyond elasticity limit; plasticity deformation level for lower storeys is higher than for upper storeys;
* seismic response for 3-layer bearing wall buildings is lower than for masonry wall buildings and at the same time they have smaller displacements than cast-in-situ reinforced concrete structures.

CONCLUSIONS

At sites as Erzincan, Leninacan and others situated on thick layers of soils RC frame buildings should not be used especially for apartment and office buildings where thick columns and beams are undesirable from architectural point of view. In these cases buildings with rigid high energy dissipation walls buildings are preferable. They are more earthquake resistant structural systems than RC frame buildings and they have also economical advantages. These buildings are: buildings with 3-layered walls with inner layer from RC or concrete and external layers from any available material including local materials as hollow bricks, limestones, hollow concrete blocks and even adobe; monolithic RC wall building; RC prefabricated large panel buildings.

The 3-layered walls buildings have some advantages comparing with other types of bearing wall buildings. Some important advantages are:
* no need to construct factories for prefabricate elements as in case of large-panel buildings;
* no need in wooden or other materials for formwork during concreting as in case of RC 1-layer monolithic walls;
* the possibility of using for external layers any available material, including local materials as hollow bricks, tuff, limestones, adobe, hollow concrete blocks a.a.

The research studies of 3LWB should be continuated and activated to decrease uncertainties in prediction of seismic behaviour and improve optimal design methods.

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