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GRAVITATIONAL SEISMIC COLLAPSE MECHANISM ANALYSIS IN VIEW OF CONCEPTUAL DESIGN

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SUMMARY

The structural collapse prevention is the most important goal of aseismic design. To prevent the collapse the actual mechanism of seismic structural collapse should be understood. In seismic analysis the horizontal seismic design load is assumed as the main part of seismic action. Actually only very seldom the horizontal load is the reason of collapse. The results of post-earthquake observations and analysis demonstrate that in most cases the reason of ordinary buildings and structures seismic collapse is the gravitational load (plus sometimes vertical seismic load) when the structure is previously damaged and deformed due to the earthquake action. The gravitational collapse model is presented in the paper. The collapse mechanisms are very complicated and very difficult for mathematical analyses. They need further investigations. But some important conceptual design rules could be and are deducted from the gravitational collapse mechanism recognition and analysis in view of seismic collapse prevention. Among them there are changes in vertical load bearing RC structural elements design, the limitation of the vertical static load compared with the vertical load bearing capacity, the crack orientation control, the development of dissipation inside bearing elements using proper relation between the block and mortar strength. One of the gravitational collapse model deductions is the universal parameter formulation to estimate the structural earthquake survivability. It is the relation between the vertical static load and vertical bearing capacity of the structural bearing system. Some conceptual approaches are proposed to plan the preferable failure mechanisms to increase the safety and seismic survivability of structures during the strong earthquakes.Using simple design means the survivability of a structure against seismic collapse could be sufficiently increased.

INTRODUCTION

The recent disastrous earthquakes - Spitak (Armenia) 1988; Erzincan (Turkey), 1992; Nortridge (California, USA), 1994; Kobe (Japan), 1995; Neftegorsk (Sakhalin), 1995, confirmed the necessity of design concepts and seismic building codes improvement (Eisenberg, 1997, A.S.Elnashai, 1996).

In spite of drastic differences of design seismic loads and conventional dead and live loads the methods of structural seismic design and design based on these conventional loads still do not differ very much. [Oskar de Buen, 1996, Housner 1982, List of World Codes].

- Design seismic loads are determined in most cases as equivalent horizontal static loads, and structural load capacity analysis is carried out as conventional strength analysis. (Eisenberg, 1997, Paulay, 1996)

- Design seismic load values are 4-10 times lower than inertial forces corresponding to design response seismic accelerations of linear structures.

- The principle of equal strength of all structural members and joints is assumed to be one of fundamental design principles. (Buen 1996, Eisenberg 1997).

According to Seismic Codes of Russia, USA, and other countries design seismic load with respect to reduction coefficients equals sometimes to less than 10% or 20 % of inertia forces corresponding to the actual acceleration.

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The ductility concept which is the background for reduction factors R values establishing sometimes works but sometimes leads to erroneous results. The ductility demands could he dozen times higher comparing with the ductility supplies according to the current Codes (Eisenberg, 1994).

The predictions of the intensity and other parameters of seismic motions given at the Seismic Zonation Maps are far from being exact. Very often the accelerations, velocities, displacements are many times higher comparing with predicted design ones.

One of the ways to increase the seismic safety and survivability of structures is to plan during design the beneficial mechanisms of seismic collapses using conceptual design approaches, and avoid the undesirable collapse mechanisms. The refusal of the equal strength of structural elements principle is one of the conditions to develop the new design concept. The analysis of actual seismic collapse mechanisms is the main way to develop this concept.

Buildings of two types collapse mostly during all recent earthquakes. They are 1. RC frame buildings without strong enough diaphragms and 2. Wall buildings of weak wall materials. The analysis of their collapse mechanisms including the analysis of the "steel flower" development in damaged RC frames was carried out.

The explanations of the "steel flower" developments given by Prof. T.Paulay and by Prof. A.Elnashai were analysed. A different explanation of "steel flower" development is presented based on the proposed model of gravitational collapse mechanism of seriously damaged, due to the multy - dimensional seismic motion, RC columns. The gravitational collapse model explains as well the cracked wall collapses.

Theoretical and experimental studies, including engineering analysis of earthquake consequences, lead to the conclusion that the local failures can play double role. They can enable, provoke the total collapse of the structure, but they can prevent the collapse.

The real mechanisms of seismic failures are analyzed to explain this aspect of local failures role.

SOME CONTRAVERSIONS BETWEEN DESIGN MATHEMATICAL MODELS AND ACTUAL FAILURE MECHANISMS.

Although heavy damages are accepted as permissible during the strongest earthquakes, and this is correctly from physical and from economical points of view, the mathematical models on which the Seismic Building Codes procedures are based and which are used in design practice are sufficiently linear and elastic.

To reconciliate this contraversion between the design philosophy and mathematical models ductility concept is used.

In the Russian Seismic Building Code the maximum design acceleration is about 0,5g. It means the seismic coefficient value about 0,5. Then some reduction factors reduce the seismic value several times. And for ordinary buildings the real seismic coefficient taking into account the reduction factor became approximately about 0,1. The reduction factors depend of the ductility factor.

Principally the same procedure is used in US: UBC, SEAOC, ATC, in Eurocode - 8 (List, 1996).

The fundamentals of the ductility concept are very simple. It is a rule for modelling an inelastic non-linear structure using linear elastic mathematical models which are much simpler and familiar to engineers.

The ductility concepts simple meaning is: it is permissible to carry out the seismic analysis of a structure using design seismic force several times lower than the actual maximum seismic forces if two conditions are fulfilled:

1. The maximum seismic displacements of the non-linear system are equal to the maximum seismic displacements of the linear system.

2. These values of displacements are not dangerous and the structure will not collapse.

The condition 1 was declared by some prominent experts (Newmark and Hall, 1992).

If the ductility factor $\mu = X_{max}/X_y$ is known, say, from experiments, the design force for structural design calculations could be defined in a simple way using condition 1. And this is the simple background for using the design forces several times lower comparing with linear system elastic response loads. But what is wrong with this approach?

It could be easily shown using the response spectra of linear and non-linear systems calculated taking as seismic inputs the instrumental strong motion accelerograms of some recent earthquakes, and then comparing the response values and the actual seismic behaviour of structures, it means, comparing the demands and the supplies. The elastic and inelastic earthquake response displacements were calculated (Eisenberg, 1994, 1995). The yielding force corresponds to seismic coefficient C = 0,1. The spectra are presented at the Figures 1a, b, c, d, e. Five instrumental accelerograms have been used as seismic inputs:

a. Armenia-Spitak earthquake, December 7,1988, the Gukasyan - town accelerogram, NS component.

b. Erzincan - Turkey earthquake, March 13, 1992, NS component.

c. Mexico Earthquake, September 17,1985, EW component.

d. Bucharest earthquake, May 4,1977.

At the Figure 1 the relation is presented of inelastic and elastic systems response displacements which have equal initial natural periods. The simple ideal bilinear inelastic model is used in the calculations and the yielding force R_y is 0,1 of the maximum elastic force corresponding the maximum displacement of the elastic system. It means that design seismic coefficient is 0,1. The horizontal line against the value

$X^{max}_{ie}/X^{max}_{e} = 1$

corresponds to the case when the "supplies" are equal to the "demands". It could be seen that they are far from being equal. The “demands” are in some cases many times higher comparing with the “supplies”.

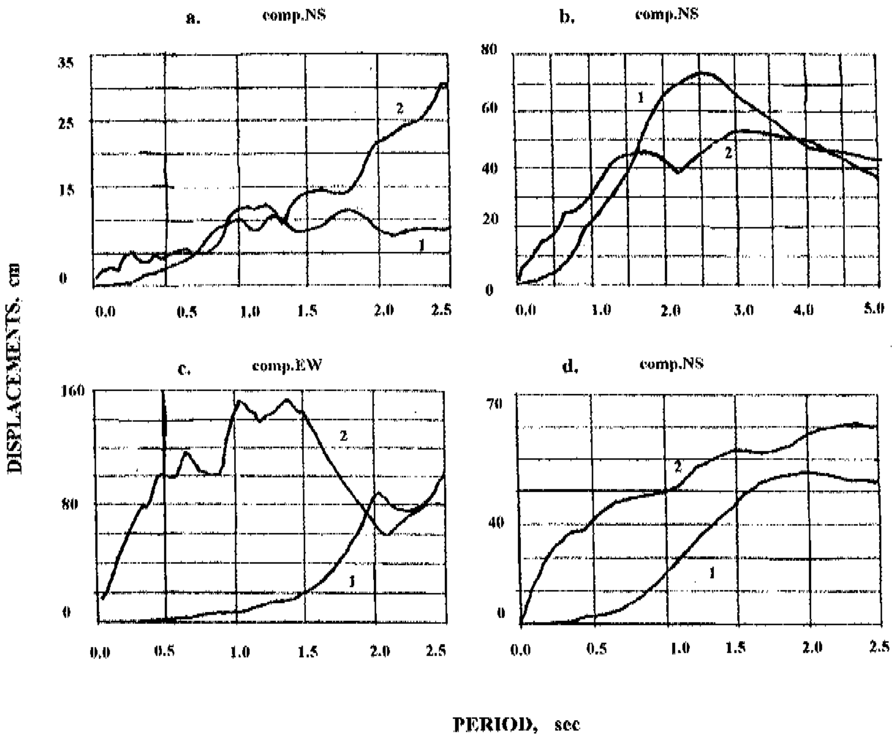

Fig. 1. The displacement response spectra of elastic (1 - C=1,0) and inelastic (2 - C=0,1) systems.

Earthquakes: a) Gukasian (Spitak); b) 1988 Erzincan; c) 1992 Mexico; d) 1985 Bucharest, 1977

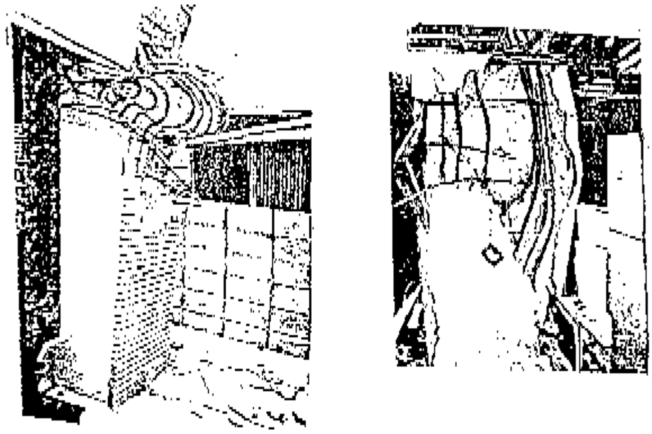

Fig. 2. RC columns damage (Kobe, 1995)

But the actual difference is even higher than that.

The natural periods of some RC frame buildings were instrumentally measured in Leninakan before the Armenia, 1988, earthquake, and after it. In some cases the values of periods after the earthquake were 3 times higher comparing with the initial periods. The reason of the increasing of periods is the cracking and damages in the structure during the earthquake. So, the horizontal displacements were many times higher comparing with design values. They reach several dozens of centimeters.

The critical P-delta displacements were compared with the actual horizontal displacements of the structures calculated using as the seismic inputs the accelerograms of Erzincan earthquake, March 13,1992. The actual displacements were much higher comparing with the P-delta critical displacements. These enormous high displacements themself are enough to explain the total collapses of the frame buildings.

Of course the concrete have crushed. The total, or almost total vertical static load was transmitted to the steel bars only.

And also the conclusion is very clear that the ductility concept sometimes does not work, and corresponding parts of the Seismic Building Codes should be improved. It is evident that for some structures time-history analysis should replace the too simplified Code approach which is based on the ductility concept.

SEISMIC FAILURE AND COLLAPSE MECHANISMS. GRAVITY COLLAPSE MECHANISM.

During Spitak-88 earthquake 98 per cents of five-story and nine-story RC concrete frame buildings collapsed. Deth - toll amounted to dozens of thousands people. The other collapsed buildings were wall buildings 2 to 5 storey of weak wall materials, brick masonry etc.

80 per cents of RC frame buildings have collapsed in Erzincan, Turkey, during the earthquake of Marth13, 1992.

During Neftegorsk-95, Sakhalin, Russia, earthquake 66 per cents of the Neftegorsk-city population died when all 17 five storey buildings with bearing walls of weak light concrete blocks collapsed.

Mostly 2 types of structural systems are responsible for the majority of people deathes during the recent strong earthquakes (Borges and Ravara, 1969, Eisenberg 1997, Housner, Jennings, 1982).

One is the RC frame building system without diaphragms or with only weak diaphragms.

The other is the wall building system of weak wall materials.

Usually the RC frame failure are accompanied with the typical "steel-flower" picture.

Reinforced concrete columns failures at different earthquakes look similarly. (Fig.2)

Why do so often reinforced concrete frame buildings totally collapse? Why are "steel flowers" as shown at Fig.2 created?

Several different explanations of deformations like that are presented in literature.

Prof. T. Paulay (Paulay, 1996) explains it in the following way. Reinforcement compression-and-extension occurs at column flexure. Extended rods of longitudinal reinforcement are being deformed - stretched in elastic and consequently in non-elastic areas. It causes their considerable elongation. Compressed rods become stretched when column movement in the opposite direction. On the contrary, rods having been stretched due to extension are being compressed as they are extended they bend and buckle. Though, this working scheme is partially true, but, probably, only to a certain extent.

First of all, rods stretching at column flexure not always are considerable enough to create such big buckled arches "leafs".

Secondly, seismic movement prevails, as a rule, in one direction, and "steel flowers" leafs are, mostly, symmetrical in relation to both orthogonal symmetry axes. T.Pauley's model does not explain such deformation type.

Prof. A. Elnashai have presented a different explanation. The typical buckling of reinforced column vertical rods is substantiated by action of intensive vertical component of seismic movement, which increase axial load sufficiently and leads to stability loss and rods buckling. Prof. Elnashai points to the above symmetrical "steel flowers" as an additional argument to substantiate the explanation.

Though A. Elnashai hypothesis in some cases may prove to be true, it does not explain the major reason of reinforced concrete columns failure and "steel flowers" of buckled rods formation.

One could observe "steel flowers" creation in cases when the role of vertical component hardly could be noticed. The similar destruction of reinforced concrete columns has occurred in Bucharest during the earthquake on May 4, 1977, when earthquake focus was rather far from Bucharest, in the mountains of Vrancha. The similar situation occurred in Mexico-city, 1985, as well as in many other cases when vertical seismic load could hardy constitute the considerable portion of gravity load of structure and other loads (Oskar de Buen 1996).

The design model and the actual mechanism of reinforced concrete columns failure should be compared to explain the failure mechanism. They differ considerably. The design model which should adequately describe the physical destruction process does not describe it, in fact.

Assumption that forces caused by both - vertical static and seismic load - are supported by two bearing components: concrete and reinforcement - is accepted for reinforced concrete column design. The following relation is used for it:

$$P = N_s + N_c \qquad (1)$$

where P is the actual axial load, static plus seismic,

N_s and N_c - reinforcement and concrete loading capacity, correspondingly, reduced by safety coefficient. Formula (1) was somewhat simplified to illustrate the matter.

In fact, a column at strong earthquake does not work in elastic phase. Depending on combination of different factors, horizontal cracks of different extention develop. In general, column and frame stiffness falls, and vibration amplitudes increase, concrete crumbles in the maximum moments zone, firstly, sometimes through the whole height, and vertical gravitational seismic load is fully transferred to longitudinal reinforcement rods. As a result, $N_c = 0$, and $N_s << P$.

Due to the fact that reinforcement rods are not designed for so high vertical forces, they loose their stability, buckle, and break stirups and transverse rods.

Hence, the main reason for the most common reinforcement concrete columns failure mechanism is action of axial vertical, gravitational and seismic, forces. Due to seismic displacements brittle concrete breaks, crumbles, and looses the ability to resist axial forces. Transverse reinforcement rods do not possess sufficient bearing capacity to resist complete axial load when concrete is not available. Stability loss due to compression and rods buckling develops with ensuing creation of space reinforcement "steel-flowers". It is one illustration of the gravity collapse model.

Let us consider another example - building walls failure.

Horizontal, vertical, oblique, cross cracks appear in walls, piers, and wall elements during strong earthquakes.

The thorough analysis leads to the conclusion that collapses of bearing walls caused by gravitational and seismic actions on walls, already damaged in the form of oblique, inclined, sometimes multiple cracks.

Total failure of seventeen large-block buildings have occurred in Neftegorsk on Sakhalin during the earthquake on May 28, 1995. Failure analysis led to the following conclusions. Due to low strength of wall block material load capacity of walls was rather low comparing with vertical load. It exceeded not very much the actual structure dead load. Epicentral area of an earthquake was several kilometers from Neftegorsk. Taking into consideration earthquake high magnitude it is clear enough that the vertical earthquake component was high, and witnesses' evidence confirmed subjects jumping up in buildings. Apparently, maximum vertical acceleration were in the range of 0,5g to 1,0g. Vertical gravitational plus seismic load has exceeded walls bearing capacity, which were damaged by earthquake. Total destruction in this situation was inevitable. Vertical load, static plus seismic, played the decisive role in building failure.

But during the Mexico earthquakes, 1957, 1962, 1985, a lot of masonry wall buildings collapsed when the vertical component of seismic load was not too high.

The reason of collapse was the dead load action upon the cracked walls. And the cracks, oblique, vertical, crossing were caused previously by the seismic load. The cracks in walls play the same role in wall collapse mechanism as concrete crumbling and cracking in RC columns.

So, in both cases, RC columns or walls, the reason and the mechanism of collapse is the same. It is the gravity load plus some vibration action on the formerly damaged vertical bearing elements which became unable to resist the gravitational loads.

SOME CONCEPTUAL DESIGN RULES RESULTING FROM GRAVITATIONAL FAILURE MODELS.

Here are shortly presented some deductions of the gravitational collapse mechanism studies in view of aseismic structural design improving.

If we assume that the reason of structural collapse is the vertical static plus seismic loads acting on the structure previously damaged by earthquake action, two main design rules should be formulated to decrease the collapse probability.

One rule is to control the damage (horizontal displacements values, cracks directions, successiveness of local damage occurring and developing).

For example, the cracks, inelastic deformations and other damage should develop, firstly, in structural elements which are not loaded with vertical loads. The “strong column-weak beem” concept is only one case of this approach. The direction of cracks is important. Inclined cracks are more dangerous comparing with horizontal or vertical cracks (Eisenberg, 1998). To use this rule in design practice the popular equal strength concept should be refused.

The second rule is the limitation during design the values of vertical static loads comparing with vertical bearing capacity.

The presenting study have brought to conclusion that the relation between the value of static vertical load on bearing elements and the value of bearing capacity of the elements is an important universal parameter which characterizes the seismic survivability of very different structural systems. The less is this parameter the higher is the survivability of a structure. A similar recommendation was formulated by other authors from another point of view (Borges and Ravara, 1969, Raizer, 1998).

CONCLUSIONS

6.1. A new gravitational mechanism of structural collapse during intensive earthquakes is presented.

6.2. Explanations of the usual forms of seismic collapses of RC frame buildings and wall buildings are presented using the gravitational collapse model.

6.3. Some conceptual design rules are formulated for total collapses of structures prevention and structural earthquake survivability increase.

6.4. If cracks and inelastic deformation are permissible they should be regulated during design because "good" and "bad" seismic mechanisms from point of view of collapse exist.

6.5. The wide distributed among engineers concept of equal strength of different parts, different elements of the structure should be rejected in case of aseismic design.

6.6. In the recently revised version of Russian Seismic Building Code different factors were recommended for different elements and sections to organize the desirable failure mechanisms.

6.7. The development of Codes and design procedures should be based on the actual seismic performance understanding and on collapse mechanisms analysis during strong earthquakes.

6.8. Further research studies of structural failure and collapse mechanisms during intensive earthquakes are necessary.

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