Seismic resistance of reinforced concrete high rise buildings structural elements

SEISMIC RESISTANCE OF REINFORCED CONCRETE HIGH RISE BUILDINGS STRUCTURAL ELEMENTS

by

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SUMMARY

Plastic design elements of reinforced concrete structures under monotonic and cycliy loading as simulation of earthquake conditions are presented. The mechanism of stiffness degrading and same principles for high-rise buildings with shear wall system are analysed. On the basis of the results of experiments coried out by the authers on large models conclusions are offered on the behavior of joints under cyclic loading.

RESUMÉ

On presente les elements du calcul plastique des section du beton armé exposés an forces monothones et cicliques somme la simulation des forces sismiques. On à donné une analise du mechanisme de la deterioration du rigidité et les principes du projet des bâtiments houts avec les mûres portants. Quelques conclusions de comportement des joints exposés an forces cicliques de resultats des experiments propres sont donnés.

ZUSAMMENFASSUNG

Die Elemente plastischer Berechnungen von Stahlbetonschnitten, welche mit nachgeahmter seizmischen Belastung einförmigen und zyklischen Belastungen ausgesetzt wurden, werden hier vorgeführt. Der Mechanismus des streifhcitsverlustes und die grundsätze der Hochbauprojektirung mit Tragwänden werden ferner analysiert. Auf grund eigener Versuchsergebnisse auf grossen Modelen werden Beschlüsse über das Betragen von Verbindungsstellen bei zyklischer Belastung dargeboten.

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1. INTRODUCTION

Reinforced concrete residential and office buildings in Yugoslavia are constructed shear wall system in both ortogonal directions with various technological procedures applied (slide form,tunnel form, large panel system etc). Skeleton structures with rigid walls or core are used infrequently while the skeletons without rigid walls are almost unknown. This is very different from the practice in the U.S.A. where the frequency of these structural system increases in reversed order.

Only after strong earthquake ground motions in Caracas in 1967 and Managua in 1972 and the damages of non-structural elements in framework structures without rigid walls, i.e. partition walls, equipments, installations and furnitures which are often more valuable than the structure itself, the necessity of limited displacement as well as a sufficient strength for a building was accepted (5). Security of this kind can be achieved by decreasing the story drift or by increasing the stiffness with reinforced concrete shear walls. Research of behaviour of shear walls under seismic forces is still under way in the world.

The reinforced concrete walls are almost always constructed mutually bound by connecting beams. As the damage of those beams in strong earthquakes does not jeopardize the stability of building as a structural assemblage for bearing vertical load, it is desirable to direct the forming of plastic hinges to the beams.

2. PLASTIC DESIGN OF REINFORCED CONCRETE UNDER SEISMIC CONDITIONS

Since the plastic design of reinforced concrete in non-seismic conditions has been introduced and applied in many countries, the basic assumptions of the design will not be elaborated herein. For seismic conditions it is necessary to extend some assumptions and separate the computation for monotonic and cyclic loading.

2.1. Monotonic loading

Stress-strain curve for concrete and steel shall be extended beyond conventional limits. After publications of numerous researchers in the period between 1928 and 1933 it is well known that the deformability of concrete with lateral deformation prevented, is several times greater than that of concrete with free lateral deformation. Should the concrete be successfully confined by using dense stirrups, the stress-strain curve for concrete will be extended several times over. The stress-strain curve thus becomes dependent on the ratio of confinement which can be expressed as a volume percentage of reinforcement by transverse bar. The analityc equations for such a diagram were given by Park (6) and are shown hereafter. Fig. 1. represents the stress-strain curve for concrete.

Region AB $\varepsilon_c \leq 0{,}002$ $\quad f_c = f'_c\left[\frac{2\varepsilon_c}{0{,}002} - \left(\frac{\varepsilon_c}{0{,}002}\right)^2\right]$

Region BC $0{,}002 \leq \varepsilon_c \leq \varepsilon_{20c}$ $\quad f_c = f'_c\left[1 - Z\,(\varepsilon_c - 0{,}002)\right]$

Region CD $\varepsilon_{20c} < \varepsilon_c$ $\quad f_c = 0{,}2\ f'_c$ where

$$Z = \frac{0,5}{\varepsilon_{50n} + \varepsilon_{50h} - 0,002} \qquad \varepsilon_{50n} = \frac{3+0,002\, f'_c}{f'_c - 1000} \qquad (f'_c \text{ u psi})$$

$$\varepsilon_{50h} = \frac{3}{4}\rho \cdot \sqrt{\frac{b''}{s}}$$

ρ is the ratio of volume of transverse reinforcement to volume of concrete core i.e. confinement ratio $\rho = A_v/b''.s$, where b" is the with of confined core, s - spacing of stirrups, A_v - area of stirrups, Z - parameter defining the slope of assmed stress-strain curve.

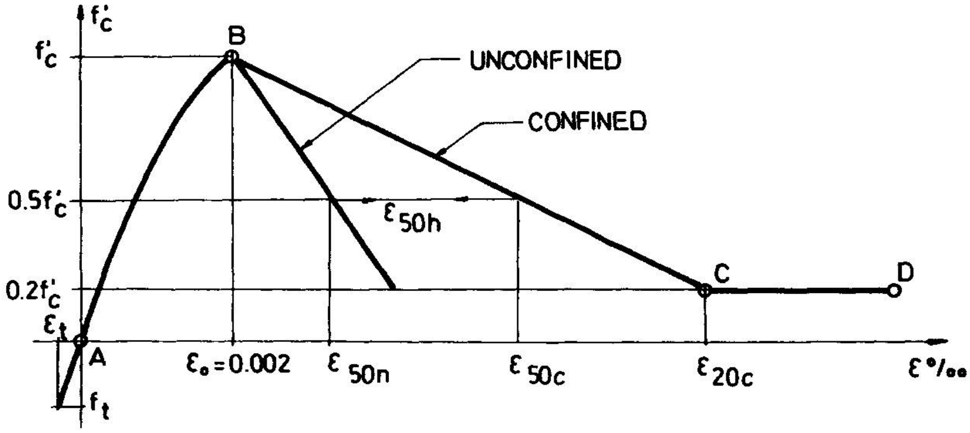

Fig. 1. Stress-strain curve for confined concrete

The stress-strain curve for steel, wich including the whole range of stress-strain relationships can be comparatively simply defined by a schematic representation of the diagram found when testing the steel. Fig. 2.

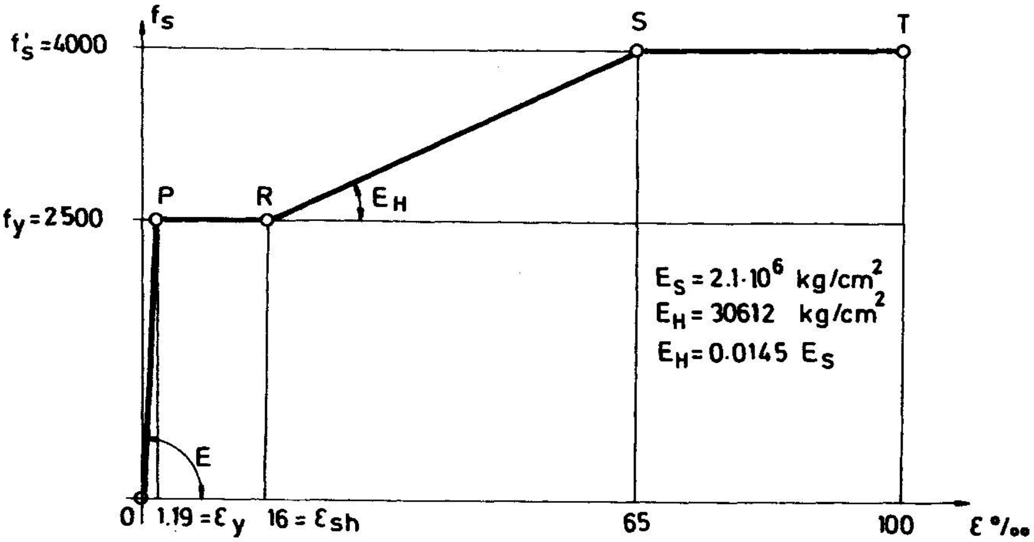

Fig.2. Idealization for stress-strain curve for steel (trilinear approximate)

For the ultimate stress and ultimate strain, the stress-strain curve for concrete and steel as shown in Fig.1 and 2. have to be used as well as the basic assumptions of the plastic design method (the tensile strength of the concrete may be neglected; Bernoulli-Navier's hypothesis of plane sections is valid for plastic range also; the equilibrium of external and internal forces shall be established). The computation of M-Ø relationship of one bending section is shown in Fig. 3.

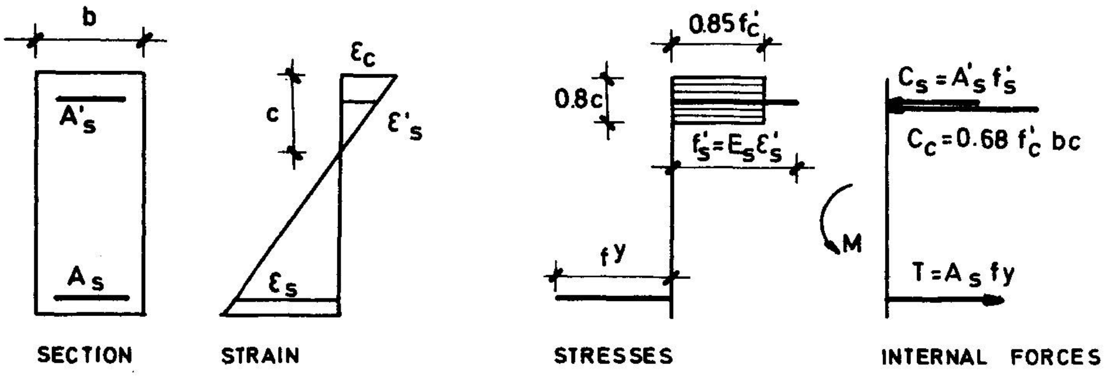

Fig. 3. Scheme for computation of a monotonously loaded reinforced concrete section

For a given value of concrete strain ε_c the value for c shall be selected and the strains for steel ε_s and ε'_s found using the diagram in Fig.2. Corresponding stresses can be obtained from stress-strain curve for concrete and steel according to Fig.1. and 2. The values of internal forces T, C_c and C_s may be determined multiplying the stress by areas. If the equilibrium is no satisfied i.e.$\Sigma H \neq 0$ and $\Sigma M \neq 0$, the computation shall be repeated with a different value for c. When the equilibrium is obtained, the curvature can be obtained from the expression $\phi = \varepsilon_c/c$. In the case of weakly and moderately reinforced sections, the relation ship M-Ø will be similar in form to the stress-strain curve for steel.

2.2. Cyclic loading

A idealized stress-strain curve for confined concrete under monotonic loading is the envelope curve of the stress-strain curves for cyclic loading. The shape of $f_c - \varepsilon_c$ curve for cyclic loading can be idealized in several ways in order to approximate actual behaviour as experimentally found by Sinha, Gerstle and Tulin (8). It is considered that a more complicated equations for representation of hysteresis loops because of a dominant influence of the steel response in hysteresis of reinforced concrete. Fig. 4 shows one possible idealized representation of the stress-strain curve of concrete with cyclic loading, according to Park (6). Analytic equations must be given for each of the region AB, BC, CD and FG, HE.

The stress-strain curve for steel under cyclic loading is a function of virgin properties of material, previous stresses, loading run number and some other factors. The first cycle after the first yield exursion, the $f_s - \varepsilon_s$ relationship becomes, due to softening of material, a flat curve with no prominent yield-point. Fig. 5. shows the idealized stress-strain curve for steel.

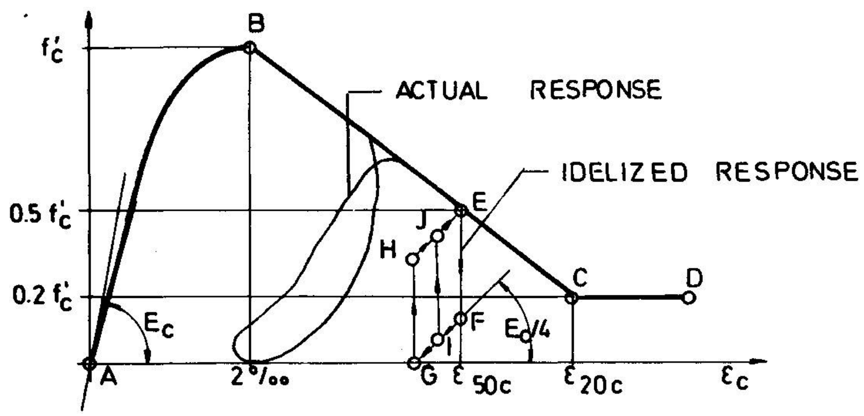

Fig. 4. Stress-strain behavior of concrete with cyclic loading

The right-hand diagram in Fig. 5. can be used as more favourable because it coresponde better with actual curve. The loading parts of the stress-strain curve is reprezented by the following form of the Ramberg-Osgood function.

$$\varepsilon_s - \varepsilon_{si} = \frac{f_s}{E_s}\left(1 + \left|\frac{f_s}{f_{ch}}\right|^{r-1}\right) \qquad \text{where}$$

ε_s - steel strain, ε_{si} - steel strain at begining of loading run, f_s - steel stress, E_s - modul of elasticity of steel, f_{ch} -characteristic stress of steel in Ramberg-Osgood function, r - loading run number.

For computer programs for cyclically loaded reinforced concrete section, it is necessary to divide the concrete section into a number of discrete elements. The points on M-Ø curves are obtained by previously described procedure of equilirium of the forces and in accordance with Fig. 5.

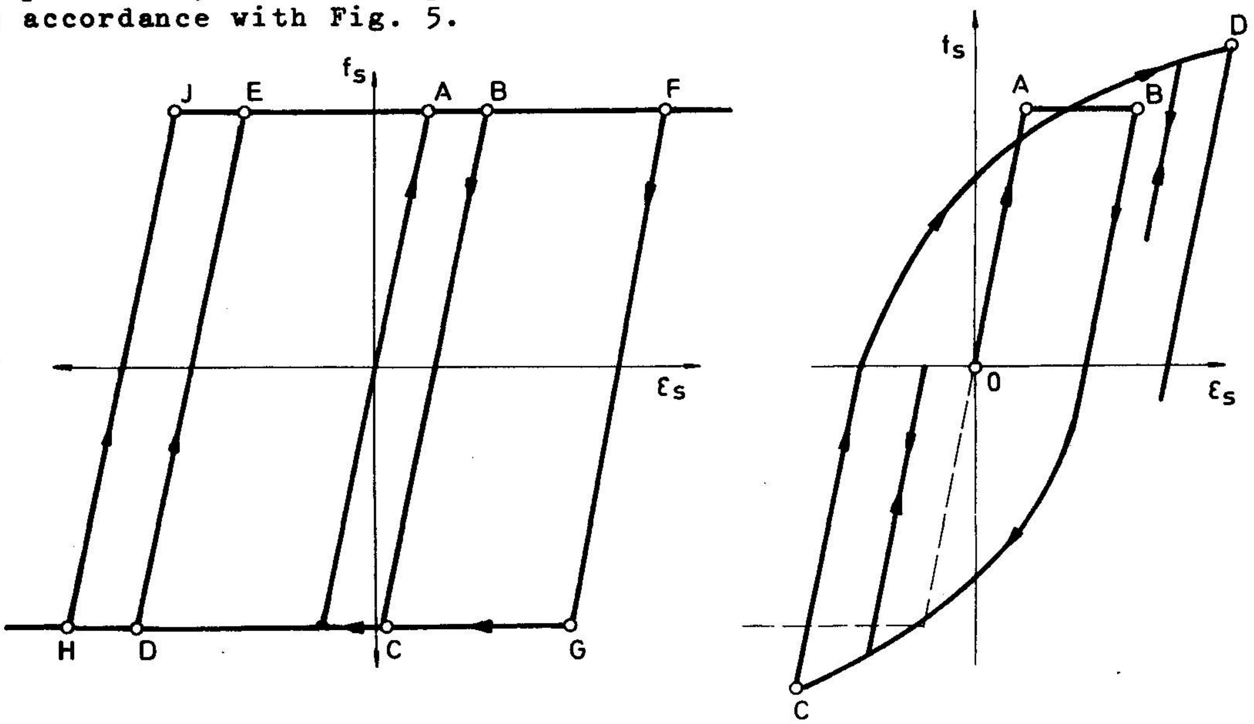

Fig. 5. Idealized stress-strain curve for steel with cyclic loading

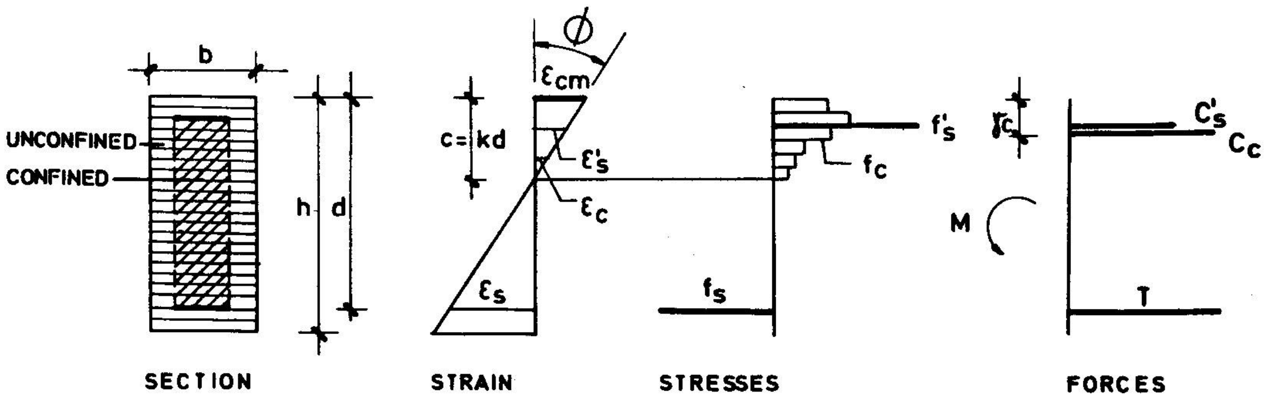

Fig.6. Schemes for computation of cyclically loaded reinforced concrete section

For each element, the corresponding f_c from assumptioned strain ε_c is determined from the stress-strain curve for concrete with cyclic loading, and the $\varepsilon_s - f_s$ relationship is found from stress-strain curve for steel. The previous state of strain should be known for each element in order to follow $f - \varepsilon$ curves. It is quite clear that such a procedure is possible only through the use of electronic computer.

3. THE MECHANISM OF STIFFNESS DEGRADING

The plastic deformations in a building exposed to strong earthquake disturbances are usually localized to certain critical region. In case of shear wall system it can be expected that the critical zones will be the sections above the foundations and the sections at places where connecting beams join the walls. In those beam sections, well-defined plastic hinges will be formed only in case of long connecting beams. If short and deep beams are constructed, which occurs more frequently, a system of diagonal cracks will appear under alternating loads due to decisive influence of transversal forces. Hysteresis loops will show instability i.e. trends of degrading stiffness. The main factors that cause that are the cracks in concrete, splitting of concrete between surface and stirrups, the steel is at the yielding stress, partial or complete cessation of reinforcement anchorage effect, simultaneous influence of shear and flexural deformations etc. Symetrically doubl reinforced section with repated cyclic loading will behave similarly to steel section represented by tensile and compressive reinforcement.

The problem of loading history arises when simulating the effects of an earthquake on a structure or structural assemblage in laboratory. Except when the experiment is carried out with a pre-programmed shaking table where a lifelike simulation is possible,the simulation of loading history for an earthquake is not easily feasible, but it is not necessary either. The testing of structural assemblage in quasy dinamic way is performed with slowly changing dynamic loading. The sequence of cycles does not conform to actual earthquake because the purpose of test is to assess total plastic capability of the assemblage tested. The usual loading history is to, increase deflections or loads as fixed and determined variables.

The reversed order can be adopted too - first the maximal deflections which then decrease. Depending on assemblage tested, different loading history will give upper or lower envelope of strength and energy dissipation (3). The total number of cycles the assemblage will be exposed to must correspond to the number which can be expected in actual earthquake. According to contemporary research, the total number of cycles which cause big plastic deformations is in the range of 20 to 100.

The loading velocity of model, in laboratory conditions always smaller than in earthquake, does not have much import on the results of quasidynamic testing. The principal effect of velocity is the increase of beasing capacity in the first cycle, i.e. first yielding of steel if load is applied very rapidly. Due to Bauschinger effect this "gain" is lostin succeding cycles and can be disregarded in elements exposed to bending (4). The effect of loading velocity for elements loaded with great lateral and axial forces has not been investigated yet.

4. SOME DESIGN PRINCIPLES FOR BUILDINGS WITH SHEAR WALL SYSTEM

The concept of ductility frame, introduced in the U.S.A. at the beginning of the sixties and widely used up to day, has proved to be unsuitable for infilled rigid walls which cannot follow big frame deformations and are thus damaged. The pure framework structure withour rigid walls is therefore considered to be relatively inadequate structural system for buildings with many nonstructural elements, which may have up to 80 % of total value with "non-structual characteristics" (5). Experience from recent earthquakes shows that buildings with shear walls have sufficient strength and considerably greater stiffness, which in turn gives necessary damage control. Ductility and energy absorption capability can be attained by connecting the walls with beams which can be made suitable for this by special reinforcement methods.

The uniform distribution of reinforcement over the whole wall region cannot be deemed correct under seismic conditions. The reinforcement should be concentrated near the edges and densely wound by stirrups in order to get confined concrete effect in compressive area. At the same time, densely placed stirrups will prevent local buckling of longitudinal reinforcement under alternating load. It is also necessary to prevent shear failure in order to get desired ductile behaviour of the wall. Therefore, sufficient number of transversal bars is required to prevent steel yielding.

The maximal resistance of walls with one row of vertical openings is attained when the structure turns into mechanism, i.e. when plastic hinges are formed, one pair in each beam and one in each wall. The sequence in which plastic hinges form depends on mutual relation of stiffness and strength of individual structural elements. It is of primary importance to attain previously determined sequence of openning of plastic hinges during the earthquake. Those joints must have great capacility for energy absorption under alternating load, without significant loss of strength. Satisfactory sequence of openning of plastic hinges is one that permits the walls to turn to plastic range only after the majority of beams already became plastic. Desired plastic properties of short and deep beams cannot be attained by conventional reinforcement, but through diagonal reinforcement in both directions. This diagonal reinforcement is constructed in the form of a basket with very dense stirrups or spiral. Experiments performed on so reinforced short beams gave very high ductility and energy absorption factors (7).

5. AUTHOR EXPERIMENTS

Laboratory tests for structural elements on ten large models of a typical high-rise building constructed by slide form system in Yugoslavia were performed. The building has nineteen storeys, with ground-plan measures 20x20 m and reinforced concrete shear walls of constant thickness of 20 cm set in two orthogonal directions. The walls are mutually connected by beams over door and window opennings and by solid slabs. The models tested were constructed to represent the parts of wall above and beneath the slab and the beam which connects them. The models were done to 1:2 scale. The procedure of testing was minutely described in other reports (1,2).

The loading history was quasidynamic with symetric cycles on x-axis. Deflections cause yielding of reinforcement immediately after the beginning and the damage of models increases. Electrohydraulic equipment has been employed during the test. The deflection was used as controlled varibale and hysteresis loops were directly registered.

The tested joints of beams and walls were damaged predominantly by bending moment because the beams were "long". Some of the conclusions drawn from analysis of experimental data are:

- long beams, conventionally reinforced, have shown high capability of plastification through forming of plastic hinges in the section at wall connection points.
- relative deformations of main reinforcement considerably overshoot conventional yield-point and can reach more than 1,0 %.
- lateral reinforcement is not strained because the beams are not working in tri-axial state of strain and the failure is caused by bending.
- the stiffness decreases with the increase of load and deformation according to adopted loading history. Substantial loss of stiffness appears when ductility ratio reaches values of more than 10.
- the collapse and real failure did not occur during the testing and so it appears that the damages can be technically easily repaired. If real building should be damaged in such a way, it's stability for vertical load would not be jeopardized.
- the beams are able to absorb considerable amount of energy, which has a favourable effect on the behaviour of building as a structural assemblage.

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