Structural engineering in earthquake zones

Main Theme C1

Structural Engineering in Earthquake Zones

Structures de génie civil en zones sismiques

Konstruktiver Ingenieurbau in Erdbebengebieten

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SUMMARY
This report is a short presentation of the practical design process for buildings in earthquake zones. It is also a call for papers dealing with problems of a seismic design, mainly the dimensioning of structures in reinforced concrete, as well as but to a lesser extent, in masonry and in steel.

RESUME
Le rapport expose le processus du projet de bâtiments situés dans des zones sismiques. Il tient lieu aussi d'appel aux communications relatives au projet, et plus particulièrement au dimensionnement de constructions en béton armé ainsi qu'en maçonnerie et en acier.

ZUSAMMENFASSUNG
Es wird kurz über den praktischen Entwurf von Bauten in Erdbebengebieten eingeführt. Der Bericht soll Beiträge hervorrufen, welche mit erdbebensicherem Entwurf zu tun haben, hauptsächlich bei der Bemessung von Stahlbetonbauten sowie Backstein- und Stahlbauten.

1. PREAMBLE

This introductory report is an attempt to make clear what kind of papers is wanted for this Session of the Congress. The Scientific Committee of the Congress has deemed appropriate to exclude papers dealing with:

- Repair, strengthening and redesign of structures damaged by earthquakes; the IABSE Venice Symposium (1983) was a better occasion.
- Description of recent earthquakes, (unless specific features of structural configuration or detailing were intended to be shown as systematically proved to be clearly advantageous or disadvantageous).
- Structural analysis under seismic conditions; the recent IABSE Structural Engineering Document entitled "Dynamic Response of R.C. Buildings" may serve several practical purposes in this respect.

On the other hand, the Scientific Committee has wished that the papers should rather concentrate on practical design problems (mainly dimensioning) under seismic conditions.

In an attempt to serve this purpose, this report starts by reminding the entire process of aseismic design. In doing so, a list of Session-topics will be built-up, except of those previously excluded. Parallelly, restating some old problems and raising some new ones is facilitated. Finally, papers are invited to offer answers to some of these design problems specifically.

2. THE PROCESS OF PRACTICAL ASEISMIC DESIGN

The following main steps may be distinguished in designing structures in earthquake zones.

2.1. Selection of the kind of materials and the structural system

After deciding the appropriate s i t e for the construction of the building, the main structural m a t e r i a l (timber, steel, masonry, reinforced concrete) is chosen and the structural system is generally decided upon: e.g., frame system, wall system, dual system.

2.2. Conceptual design, a very important step in the overall design: Structural c o n f i g u r a t i o n and empirically selected arrangement and rough dimensioning of building elements are carried-out, subject to analytical verification.

2.3. Assessment of the seismic conditions, usually expressed by a seismicity estimator, e.g. an effective peak ground acceleration max a_g and an assessment of local soil conditions (selection of a site coefficient "S").

2.4. Estimation of the natural period of vibration of the building as it has been conceptually designed. Only rough estimates are needed, in order to read normalised response accelerations ("α_T") out of a given design response-spectrum.

2.5. Evaluation of a base-shear-coefficint as specified by Codes. To mention the example of CEB Seismic Annex (1982), this coefficient may be expressed as

$$c = \frac{\gamma_n \alpha_T}{K} . S . \max a_g$$

a_g = effective peak ground acceleration

α_T = normalised response acceleration, a function of the natural period of vibration of the building; α_T is read-out of a locally valid design spectrum (usually given by Codes).

γ_n = partial safety factor, modifying the target failure probability as a function of the i m p o r t a n c e of the building,

K = behaviour factor, an overall empirical modification of the elastic to elastoplastic model (accounting for the d u c t i l i t y of the structure).

S = site coefficient (soil type), expressing the higher vulnerability soft soils impart to flexible buildings.

2.6. Selection of the structural model: The structural analysis under seismic actions is to be made by means of a more or less simplified model simulating the dynamic behaviour of the actual building (linear elastic models are used in the majority of normal buildings).

2.7. Design load combination, which takes into account the reduced probability of variable actions to be simultaneously present with the design earthquake actions.

2.8. Structural analysis (modal analysis, equivalent static analysis).

2.9. Check for the compliance of Design Requirements. In this respect, it is important to underline the significance of performance-oriented modern Codes, which make clear to the designer **requirements** and **criteria** to meet these requirements.

a) Safety requirement, fulfilled by the following means:

- Stability of the building as a rigid body, and foundations' stability.
- Control of failure mechanisms; normaly, it suffices to ensure that plastic hinges will first appear to beams than to columns ("capacity design").
- Check of the ultimate capacity of critical regions of all building elements, versus the action-effects found by the structural analysis.
- Care for appropriate qualities of materials and, mainly, for appropriate detailing, in order to ensure sufficient d u c t i l i t y - a fundamental property of an earthquake resistant structure.
- Quality assurance plans, being extremely more needed in earthquake situations than in any other case.

b) Limit deformations - a requirement aiming at the limitation of d a m a g e s and misfunctioning of the secondary (and most expensive) organism of the building.

3. IDENTIFICATION OF PRACTICAL PROBLEMS and CALL FOR PAPERS

Following the guide-lines set forth by the Scientific Committee of this Congress, the Session this Introductory Report is intended to serve, should focus on p r a c t i c a l problems in designing structures under seismic conditions. Among these problems conception and dimensioning seem to have a preference, although some everyday problems of structural analysis would also be considered as "practical" as well.

Under this optics of priority, this part of the Report is an attempt to reiterate the importance of some problems and to invite research workers and designers to offer their knowledge and experience contributing to the improvement of the solutions already given to these problems. Of course, the selection of these problems is to a certain degree arbitrary and it cannot be restrictive at all; however, if a collective effort is concentrated to certain problems only, some sound conclusions would be drawn out of this Session of the Congress, to the benefit of the profession at large.

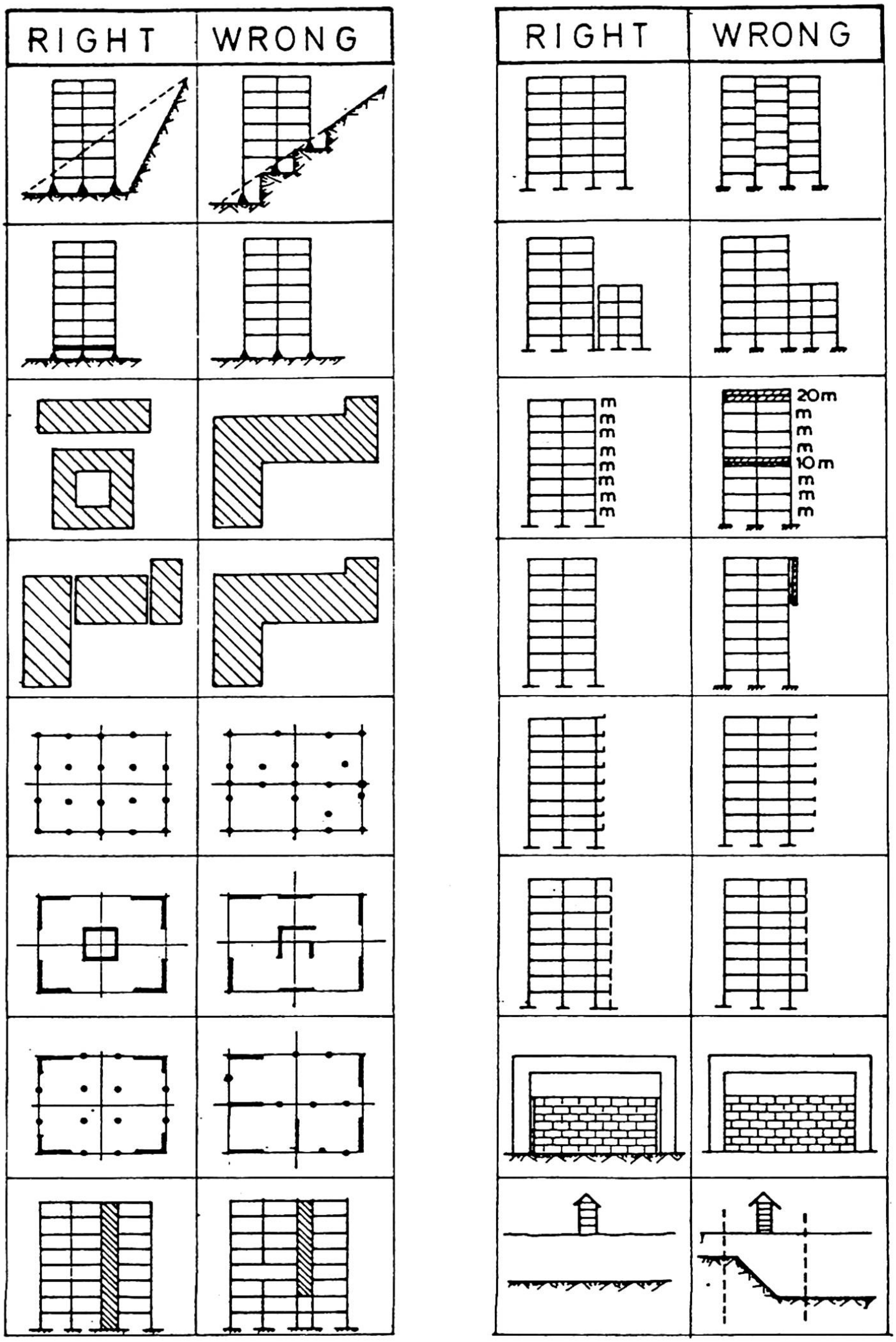


Fig. 1: Some of the rules related to the configuration of buildings in seismic zones

3.1. Structural configuration

Several Codes and textbooks provide restrictions regarding structural configuration, both in plan and in elevation; an example of those restrictions for R.C. buildings is given in Fig. 1. Some of these empirical rules are merely dictated by economical reasons against disproportionate structural costs when designing versus earthquakes. However, in their majority they are supposed to be prerequisites for the validity of the analytical models used. In fact, under the real extreme conditions of the "design-earthquake", large post-yield excursions are expected in several critical regions of the structure. Due to a large number of parameters influencing the plastic behaviour of these regions, very large **uncertainties** must be expected, which might harm (to an unknown degree) the reliability of the models used both for the analysis and for the dimensioning. Therefore, extensive asymmetries and or large non-uniformity of mass-distribution or stiffness-distribution along the structure, may render its seismic behaviour almost unpredictable. By way of example it is very doubtfull if there is any practical possibility to impart to the sections AA and BB of Fig. 2 their extremely high ductility demand.

This being said, the question arises "how much realistic are the specific q u a n t i t a t i v e restrictions set forth by the Codes" in this respect. In other words, it would be very instructive for the practical design if a rational reassessment of these rules could be carried-out, e.g. by means of several parametric studies or even on the basis of a large experience gained possibly during real earthquakes.

3.2. Stiffness of R.C. building elements, versus real action-effects under seismic loading

To the opinion of this reporter, this is in fact an everyday design problem independent of the specific structural analysis method used. Three particular cases are considered here-below, connected to this problem.

a) Justified guide-lines are needed for the selection of stiffness of the members of R.C. frame systems. Gross-section stiffness for columns is generally used, whereas beams are occasionaly considered at cracked stage.

b) A specific problem of a similar nature is raised when, applying the equivalent static planar analysis, t o r s i o n a l effects are to be introduced. If the bearing system comprises structural walls as well, the usual assumption of "column" doubly fixed at the levels of consecutive slabs is no more valid. Justified, relatively simple, artifices for hand-made calculations are welcome.

c) Finally, stiffness characteristics of coupled-walls are strongly dependent on a x i a l load (Fig. 3). Practical rules are needed both for flexural and shear stiffnesses. Systematic experimental findings in this respect will be much appreciated.

3.3. Shear strength of short columns

There is sufficient experimental and theoretical evidence (see i.a. CEB Bull. 161/1983) on the drastic reduction of both bearing capacity and ductility of relatively short columns (Fig. 4). The full M, N, V interaction proves to be very critical in case of low shear ratio values; in fact, for $\alpha_s = M : Vd$ lower than say 4, uncoupling of dimensioning for shear and dimensioning for axial actions is no more valid, even under monotonic conditions. Cyclic actions are accentuating these phenomena.

Inspite this fact, practical design rules regarding short columns are not yet included in codes. In view of the very many incidents of such building-elements (e.g. Fig. 5), further theoretical and experimental research is needed and, above all, practical guide-lines regarding the necessary modifications to be made to conventional dimensioning-methods and detailing rules of R.C. short columns under

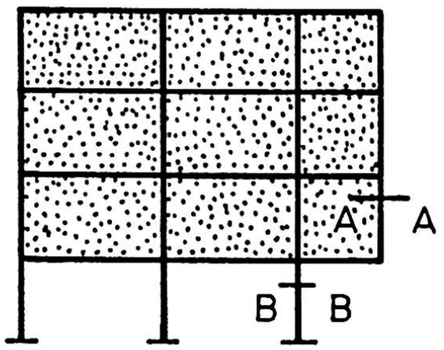


Fig. 2: This kind of structural configuration is not covered by aseismic codes; the very high strength and ductility demands of sections AA, BB reduce considerably the reliability of simple analytical models

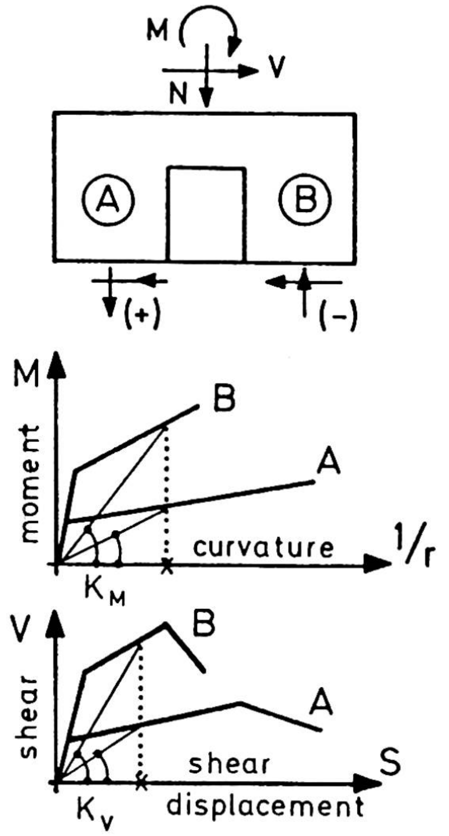
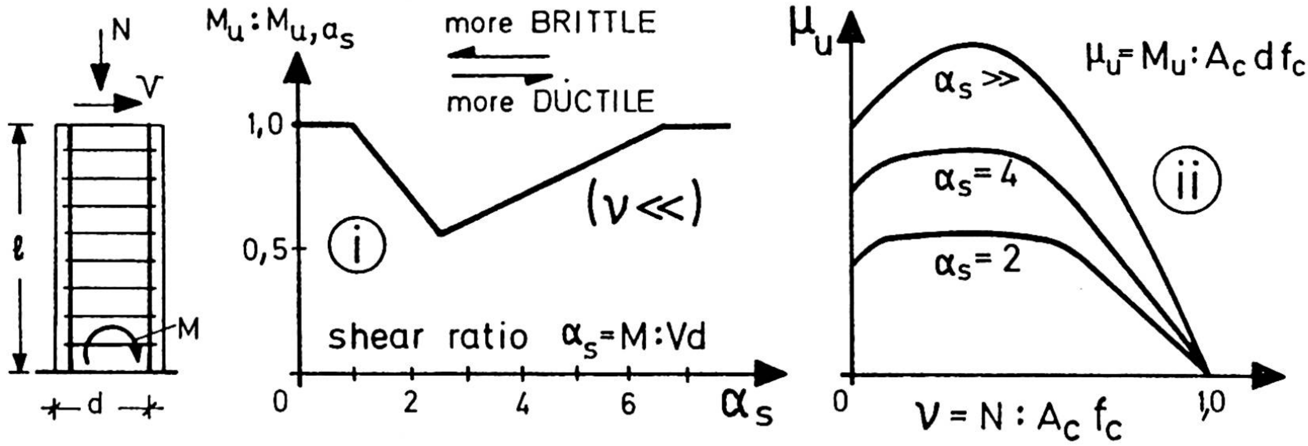


Fig. 3: Stiffness characteristics of structural walls under compression (B) and tension (A) are very much different



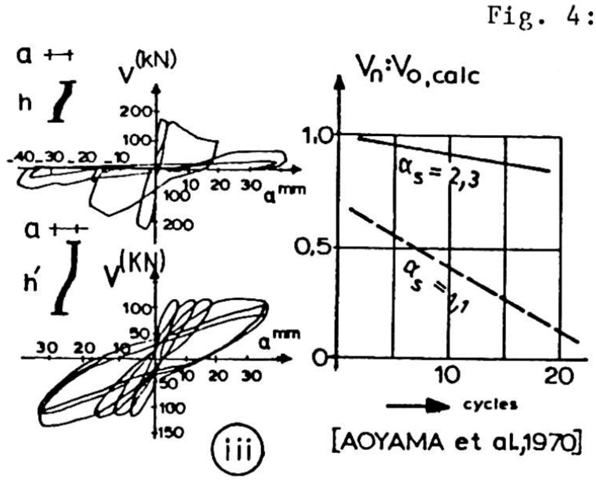


Fig. 4: Short columns (low shear ratio) exhibit:
(i) lower flexural strength
(ii) more flat M-N interaction curve
(iii) more rapid response degradation under cyclic loading
(iv) lower damping and more brittle failures

fully reversed cyclic actions. Squat-walls may be considered as presenting similar problems, and they also may be examined under this same topic.

3.4. The vulnerability of column-beam joints

One of the most vulnerable regions of R.C. frame structures in seismic situations is the column-beam joints. Their importance has been underestimated for many decades. Even now, aseismic codes provisions regarding design of joints are not internationally uniform. The complexity of the performance of these joints cannot be overemphasised.

In fact, the overall behaviour of such a joint depends on the behaviour:

a) of the anchorage of longitudinal bars of the beams, under cyclic pullouts/push-ins.

b) of the integrity of the core of the joint itself, under cyclic shear conditions.

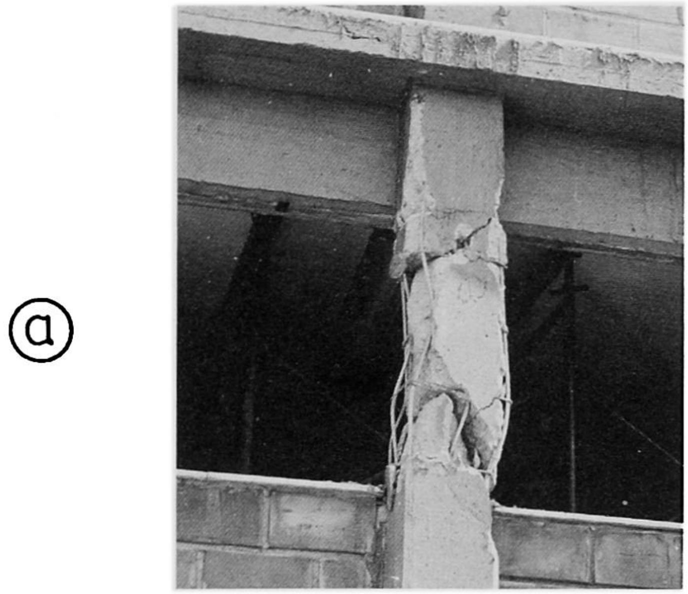
Fig. 6 is an attempt to summarise in a very schematic way the interaction between these two behaviours. Under moderate seismic conditions (or in case structural walls do not allow large displacements leading to large reversions of beam-end moments) compressive forces may be transferred through concrete after full closure of tensile cracks created by the previous cycle (Fig. 6b). In such a case, equilibrium of the longitudinal bars of the beam may be secured thanks to sufficient bond developed within the joint; transversal compression (due to the axial load of the column) is very favourable for the satisfaction of the relatively large bond demands, inspite bond degradation due to cyclic actions (compare: CEB Bull. 131, p. 74). On the other hand, shear transfer through the core of the joint is secured by the diagonal concrete **strut**, (since on each end of the strut there are available components C_b and C_c to create diagonal compression). For such a situation, two favourable consequences are derived for design: Bond may be secured without excessive additional measures, and truss mechanisms for shear transfer through the joint are not very pronounced; therefore low percentages of shear reinforcement in the joint-core are required.

However, for more severe conditions i.e. if very large displacements are imposed, without considerable redistributions of action-effects, and if a large number of full reversals is applied cyclically, tensile cracks at the beam-joint interface may not close during the next cycle; thus (Fig. 6c) compressive forces at the beam-end will be transferred to the joint only by means of reinforcement. Therefore, a cyclic pullout/push-in condition of the longitudinal bars will lead to the following doubly unfavourable result:

- Due to large reversed slips, bond degradation will rapidly take place.
- The axial force in the longitudinal bar is now almost two times higher than in the previous situations.

As a consequence, yield penetration will be rapid and the locally demanded bond (length KL in Fig. 6c) perhaps higher than available. On the other hand, since the horizontal compressive force C_b no more exists, the diagonal strut transfer of shear in the joint is alleviated and **truss** action is needed to this purpose; this action is further aggravated due to the c o n c e n t r a t e d bond forces. It becomes then apparent that if such extremely unfavourable conditions are expected, very drastic measures should be taken when designing joints: Very small diameters will be allowed for longitudinal bars, and considerable shear reinforcement will be needed in the core of the joint.

It is hoped that papers submitted in this Session of the Congress will offer criteria to assist the designer to select between the first and the second approach (which, roughly speaking, correspond to american and newzealand authors, respectively).



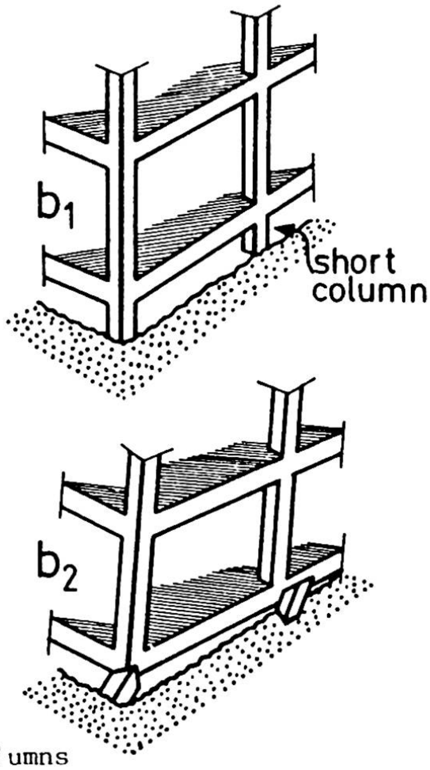
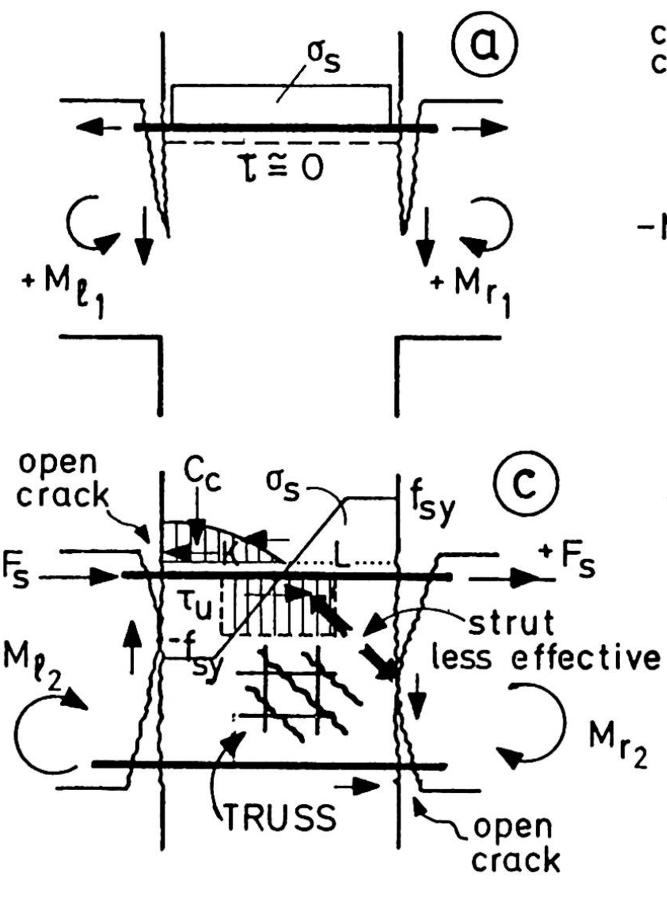


Fig. 5:

Very frequent incidents are observed on short R.C. columns subjected to earthquakes

a) The effective length of this column is drastically reduced due to the infill brick masonry
b) Typical damage in El Asnam: Failure of short columns (l = 700 mm) and almost vertical displacement of buildings



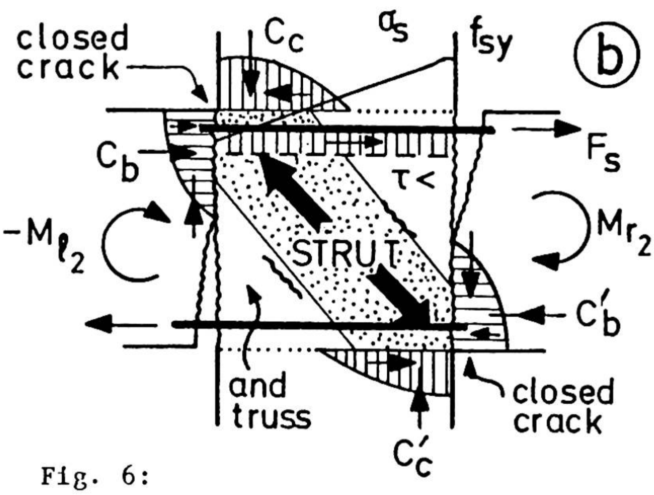


Fig. 6:

Column-beam joint (schematically)

a) Under vertical loads only
b) Under large lateral loads; previous tensile cracks at beam-ends do close during moment reversal
c) Under large lateral loads, very large displacements and large number of full reversals, previous tensile cracks at beam-ends possibly remain open during moment-cycling

3.5. Actual ductility of prestressed concrete critical regions under cyclic actions

Further evidence is needed regarding the available ductility of prestressed beam-end sections and the means of increasing this ductility. Existing code restrictions on this subject, as well as some indications that concrete confinement may not be as effective when large prestress forces are present, have produced certain difficulties in applying prestressed concrete in normal buildings situated in highly seismic zones.

3.6. Masonry

Due to the high sensitivity of beam-column joints in R.C. sway frames, reinforced masonry has recently regained its practical importance for safe low-cost housing. Full scale experiments and/or theoretical investigations on reinforced masonry walls, under fully reversed large cyclic displacements, are not as frequent as in the case of R.C. sub-assemblages.

It would be desirable to have the opportunity to read and discuss papers on this subject during this Congress. Unreinforced masonry containing appropriate strengthening-belts and ties made of R.C. are also meant to be included in this topic.

3.7. Connections of steel members

Inspite the high strength and ductility of steel per se, there is still space for additional experimental and theoretical research regarding force response and ductility characteristics of **connections** between steel elements (and specifically of beam-to-column **joints**) under large plastic reversals.

Papers on these subjects are welcome, for a better understanding of the related phenomena, in the hope to allow for less conservative design provisions.

3.8. Construction problems

Due to the additional Code requirements regarding detailing and quality of materials and workmanship used in aseismic structures, several **new** problems have to be faced during the construction in earthquake zones. Their consequences on the final performance of these structures are expected to be much more acute than in normal construction.

A couple of characteristic problems only will be mentioned here.

a) **Hoops** foreseen in the critical regions of a R.C. column (top and bottom areas) should also be provided in the core of the column-beam joint. Therefore, special techniques should be used in order to install the ready-made reinforcements of the adjacent beams. Papers dealing with solutions of this kind of problems of industrialisation of reinforcements, given in real constructions, will be useful.

b) Which specific **inspection** formats have been implemented in large and in medium size construction sites, in earthquake zones? Which are the additional organizational efforts needed and the conclusions drawn, given the sensitivity of aseismic structures versus quality drawbacks, even in the smallest detail.

Are they any field observations regarding detrimental effects of **misuse** of aseismic buildings?

Such may be the subjects of another category of papers invited in this Session.

4. CONCLUDING REMARK

It is worth to repeat here that the more or less arbitrary selection of topics proposed in the previous chapter, is by no means restrictive for the papers to be discussed during this Session of the Congress. However, if for some of these topics a concentrated effort could be given, possibly better results might be expected.

This Reporter feels already indebted to the Contributors of this Session.

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Notations

a = top displacement of R.C. column
α_s = M ; Nd =l : d shear ratio
A_c = concrete section
C_b = internal compressive force in beam
C_c = internal compressive force in column
d = height of R.C. section
f_c = concrete compression strength
F_s = steel force
l = length of cantilever column
μ, ν = normalised bending moment and axial compressive force
M = flexural moment
N = axial force
$\frac{1}{r}$ = curvature
s = shear displacement of R.C. wall
σ_s = steel stress
τ = bond stress
τ_u = ultimate bond stress
V = shear force
V_n = shear force response after "n" cycles of displacement-controlled reversals