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SF-C

Session Report
DISCUSSION OF SPECIAL THEME SESSION SF: INELASTIC BEHAVIOR AND MODELING OF CONCRETE STRUCTURAL COMPONENTS UNDER MULTI-DIRECTIONAL SEISMIC FORCES

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To capture 'alive' the interesting and fruitful discussions that followed the presentation of one Introductory Report, 2 State-of-the-Art Reports and 8 papers in the special theme session SF, the session reporters have decided to quote in verbatim some important points discussed. Due to insufficient time to verify the transcription taken from recorded tapes, the session reporters would like to apologize for any misunderstood point that may be contained in the transcribed discussions below. The session reporters would be happy if the discussion herein among the participants of this session is spred to readers of these proceedings.

DISCUSSION ON

Introductory Report: Inelastic Response of 3-D Structures and Multi-directional Seismic Forces on Structural Components

and

Sub-Theme 1: Inelastic Behavior and Modeling of Columns under Biaxial or Varying Axial Forces for Earthquake Response Analysis

H. IEMURA (Kyoto Univ., Japan) to LI, K.N. et al
1) What is the relation between axial load (N) and bending moment(M) in the loading path?
2) Is the relation between N and M determined in the 5-Spring model?
3) Can the model be used for arbitrary N-M loading path?

LI K. N. and S. OTANI (Univ. of Tokyo, Japan)
1) We assumed that axial load is directly related to overturning moment, and overturning moment is approximately related to base shear. Therefore, we decided that axial load is to vary with shear force in the column.
2)+3) The relation between spring force and applied axial load is included in the analysis. In the analysis, input information is applied to lateral deformation and also axial load. Therefore, we could use the five-spring model to any loading condition.

T. PAULAY (Univ. of Canterbury, New Zealand)
Instead of looking at failure of individual columns when subjected to various axial load demands, we should perhaps look at the global response of the structure and the role of individual columns in that global response.

S. MAHIN (Univ. of California at Berkeley, U.S.A.)
I think Tom Paulay's point is well taken in that the best way to solve a problem is to avoid it in the first place. Achieving a strong column weak girder is somewhat difficult given all the uncertainties. The intent of what we are doing is to serve as a little bit of a warning that there may be a surprise if we cut our safety margin down too close. Detailed analyses serve as good parametric studies.

S. OTANI
A preliminary analysis of 8-story frame building indicates that:
1) overall structural response such as story displacements and shears was not affected by the interaction of axial and bi-directional bending resistance,
2) the ductility demand was higher in tension side exterior columns because the yield deformation was reduced with tension load.

A. W. SADEK (Cairo Univ., Egypt) to SF-RI
1) Based on my research in the area of inelastic modelling of columns, I found that the global overall response of a building significantly depends on the specifics of inelastic modelling of columns in case of short structural periods; however, long period structures seem to be insensitive to these specifics. Please comment on that.
2) I believe that detailed inelastic modelling of R/C columns is particularly needed for explaining observed damage. However, I do not expect such refined modelling to be included in an overall analysis of buildings. I'll be glad to hear more from you on that matter.

S. MAHIN
There's been quite a few observation that there is, in general, little difference in long-period structures independent of how you model the structure. There is, however, a bit of evidences from our work and others that the prediction of the nature of local damage is sensitive to modeling. The use of refined inelastic modeling is a question of keeping a balance between the level of analysis and the objectives of analysis.

E. POPOV (University of California at Berkeley, U.S.A.)
It is encouraging to see good comparison between experimental work and analysis, but we may be creating or allowing problems to occur. We need to stop the problems by communicating better with practicing engineers/architects.

A. SHIBATA (Tohoku Univ., Japan)
Codes include provisions for eccentricity, and I agree that engineers/architects should be educated to avoid this problem.

T. P. TASSIOS (Nat. Tech. University Athens, Greece)
Prof. Otani was optimistic in stating that in the more compressed columns, ductility demand is decreasing as also the available ductility is decreasing. However, both Mr. Li and Prof. Mahin were a bit more pessimistic about that. For the sake of calibrating our codes, could you give us your advise on this issue? And, more generally, what is the road to follow in the near future in order to find available margins of safety v.s. actual uniaxial design approach.

V. BERTERO (Univ. of California at Berkeley, U.S.A.)
(comments to Prof. Paulay and Tassions' questions)
The sophisticated models presented for biaxial bending moments and axial forces are needed to carry out sensitivity analysis of these moments and forces; and find out reliable values that designers can use in their design, i.e., values of redistribution that should be considered in the final detailing of the critical regions of members. There is a need to see how the above effects can affect redistribution of shear forces demands and the supplied shear strength.

D. P. ABRAMS (Univ. of Illinois at Urbana-Champaign, U.S.A.)
Distribution of story shear is indeed related to the pattern of axial force at the base of a slender multistory frame structure. A series of tests of RC columns subjected to reversals of axial force and lateral deflection demonstrated this clearly. However, with shear attraction, one must also consider the variation in shear strength with changes in axial compression. For some specimens, diagonal tension cracking was associated with reductions in axial compression because of decreased shear strength; whereas for others, diagonal cracking was associated with increases in axial compression because of increased shear force.

S. OTANI
The proper objective of our testing is that we want to develop some mathematical models so that we can analyze structures under bi-directional motions.
We haven't have yet that good model to include interaction between axial load and bi-directional bending. But, this is just the start . . .

R. PARK (Univ. of Canterbury, New Zealand)
Moment-cuvature behaviour can be modelled very sophisticatedly by the fiber method, but it is more difficult to include the effect of deformations due to diagonal tension cracking and bond deterioration. These additional factors can have a very significant effect on the relationship between displacement response of building and the curvature demand at critical regions. What methods have you used to model these additional factors?

S. MAHIN
Use analysis to understand behavior which will lead to simplified approaches useful in design/analysis. We looked at elements today, . . . we need to consider the global structural response.

DISCUSSION ON

Sub-Theme 2: Inelastic Behavior of Three Dimensional Structural Joints

J. K. WIGHT (Univ. of Michigan, U.S.A.) to SF-R2 and G.N.GUIMARAES et al
Professor Paulay criticized the use of empirical rules for the design of beam-column connections. However, I believe the specimens tested at the Univ. of Texas were designed using such empirical rules and they demonstrated good behavior. I would like to have Prof. Kreger discuss the design of the Texas test specimens and have Professor Paulay give additional clarification of his statement.

T. PAULAY
I personally believe that shear stress itself is a meaningless quantity. If a joint is to fail in shear, it is either going to fail in diagonal compression(because cracked concrete under reversed loading cannot sustain the diagonal compression field) or it is going to fail by diagonal tension (because stirrups yield extensively and we got a corner-to-corner crack).
As far as performance related to the Texas test is concerned, my observation is that it is not sufficient to develop a certain level of strength. Yes, it can be developed but at the expense of very large displacement. Typically, the tests which have been shown followed more or less a linear increase of strength up to a drift of 1.5 -2.0 %. Well, these tests indicate that a frame or a sub-assembly is much more flexible that any assumption that has been used in normal frame analysis.

M. E. KREGER
We proportioned the specimens to satisfy the requirements of ACI 352. One of our primary concern, especially with the high-strength specimen, was to determine

whether the empirical provisions of ACI 352 establishing the shear strength of the joints are sufficient. We proportioned the flexural strength of the beams and columns so that we could generate shear failure in the joints. As I see it, there are 3 different types of tests that you could perform on joints. One is to proportion specimens so that the joint fail in shear. Another is to have proportions so as to delay deterioration of joint and develop some type of bond stresses. The other is to design a joint that will be a representative of U.S. building construction in seismic zone and then test that joint to demonstrate the overall behavior. We are interested really in that sort of a proof test. But I don't think our high-strength specimens were exactly or inexactly what would be a well-proportioned joint. That is also true for the normal-strength specimens.

W. G. CORLEY (Construction Technology Laboratories, U.S.A.)
Would it be feasible to reduce cross ties in interior joints to cut down congestion and ease placements of concrete?

T. PAULAY
I fully agree with Dr.Corley that every attempt should be made to reduce congestion. I believe that the elimination of cross ties or intermediate ties may not be the best solution, because we do have a series of problems of bond. Some innovative solutions could be used. For instance, joint reinforcement could be put outside if you happen to form haunches.

S. MORITA (Kyoto Univ., Japan) to SF-R2
Is your model for the contribution of the slab reinforcement applicable to model the behavior in multibay structures.
Specimens in which half of the slabs is cut diagonally are satisfactory or not to examine the beam slab-column subassemblages.

T. PAULAY
Slab contribution can be interpreted in 2 ways. I prefer to use the concept of tension in the tension flange which is eventually transfered by diagonal compression to the joint. Because beam moments and column moments must balance each other, and the only place where they can do so at the beam-columns joints. Another model which is proposed by others results from the recognition that a beam must grow, plastic hinges form, so the beam becomes longer. So, if we do have a multibay structure, all the spans will eventually become longer. Everything will be subjected to tension and put the slab in compression. One of my approaches is that axial compression will on one side increase flexural strength of the beam. Inelastic deformation in steel bars never recover fully especially if you have got plastic hinges in the spans. I believe that this mechanism or the increase of flexural strength as a result of slab tension is equally applicable for multibay structure.

M. KREGER
The reason for cutting the corners off the slab is so that it will fit through slab. One thing that is very interesting from the 2 series of tests was the interaction between the transverse beams and the slab. The transverse beam was making the slab more effective. These all tied up together with Prof. Paulay's model. The twisting of the transverse beam which is pulling on the slab and the forces are being redirected back toward the connection.

S.OTANI
One comment is from the test of BRI in Tsukuba Japan on the US-JAPAN full-scale R/C test. We observed the expansion of yielding in slab reinforcement to almost full width of the span at very large deformation. So, that can be observed in real structure, too.

S. OTANI (Univ. of Tokyo, Japan) to SF-R2
The slab truss model assumes that the slab reinforcement tensile stress should be transfered to the beam-column joint by the diagonal strut. In this model, the tensile stress in slab reinforcement between the diagonal to the face of the transverse beam must be constant.
Was the strain in slab reinforcement observed in tests to confirm the validity of the model?

T. PAULAY
I didn't consider any strain compatibility. What was particularly interesting is that in the unit that we tested, the bars parallel to the loaded beam have all yielded. And the bars transverse to the loaded beam (which we put in for the sake of simulation) were hardly stressed, but have also been yielded in some cases of very high stresses, indicating that diagonal compression field must have developed to bring the flexural strength into the joints.

T. P. TASSIOS (Nat. Tech. Univ. Athens, Greece)
1. (to O.JOH et al) Could you elaborate on your statement on the possibility to improve the performance of a joint by increasing its horizontal reinforcement. Because, if this is so, this could mean that truss and/or confinement model make sense.
2. (to K.KITAYAMA et al) It seems that you reached ductility displacement factor as large as 4, but with very pinched hysteretic loops: e.g., going toward R = 1/25 you pass through R = 1/50 having lost 50% of your strength...
3. (to G.N.GUIMARAES et al) Your displacement ductility factors imposed were not larger than about 2; strut model under these condition might be fully valid. But how you could achieve much higher imposed ductility factors without increasing the horizontal reinforcement of joint (thus accepting a model other than just diagonal strut).

O. JOH (Hokkaido Univ., Japan)
I don't want to provide high-strength steel in joints. But, my test results showed that ductility factor is improved by providing high-strength steel or high amount of steel. That does not mean that joint shear strength does not increased.

M. E. KREGER
We conducted the tests without the intent of attacking the joints. Because of that, we have very high flexural ratios in all of the members. If we reduce the reinforcement ratios to something more representative of actual design, I think that we would have demonstrated much larger ductility factor. How much larger? It would be interesting to conduct the test.

T. PAULAY
As you have seen from all the presentations, displacements of 3% to 5% could be attained in the tests with full strength. One school believes that it doesn't really matter whether we talk about ductility of 2 or 3. From 3 to 4% interstory drift could be developed, and we don't want any more than that. Some people use specimens that achieve ductility of 3 at 4% displacement, while some other tests would have ductility of 6 at 4% displacement.
So, the argument is really the definition of yield displacement.

H. AOYAMA (Univ. of Tokyo, Japan) to SF-R2
As to eccentric joint, Japanese experience of 1968 Tokachioki earthquake showed that torsion (or twist) that would be caused by eccentric joint would be prevented by floor slabs if the eccentricity is outward. But it would not be prevented, hence would lead to structural distress, if the eccentricity is inward. I wonder if Prof. Paulay is going to elaborate in this type of joints, and if so, whether he has considered the above point or not.

T. PAULAY
Eccentric joints of that kind do occur everywhere and I would be perfectly honest to admit that we don't know how to deal with it. We felt that we could have such an eccentric beam-column joint. Consider the joint only with that kind of a column which is more or less the same as the width of the beam, and disregard the rest of the column, and place in all the joint shear reinforcement. But, we don't have the experiemental test. But it is important to draw the designer's attention that eccentric joints do exist and he should at least think about it. Torsion or twist in a joint could be a problem, and the role of slab is undoubtedly very important in preventing this twist.

E. DEL VALLE (Natl. Univ. of Mexico) to SF-R2
Most of the tests in column-beam joints are made restricting horizontal displacements of the column (because tests are easier that way); however, for large ductilities (and large lateral displacements) the P-Δ effects might reduce the capacities determined in the tests. Could you comment on that, please?

T. PAULAY
The P-Δ effect in these tests is simply a static situation. Most people prefer to load beam that way because it is so much easier and many laboratories lack the facilities. Based on appropriate corrections, beam deflections can be translated into story displacements which give you Δ. Knowing P-Δ, no matter what, the assembly has to resist this P-Δ and what is left is available for earthquake resistance. But, this is not a beam-column joint problem because P-Δ effect is controlled by the entire structure.

R. LEON (Univ. of Minnesota, U.S.A.)
The speaker implied that the behavior of two specimens with different anchorage lengths performed similarly; however, the amount of damage shown for the specimen with short anchorage and the amount of column rotation indicate a column failure rather than a bond failure. Can you comment on this issue?

S. OTANI
Two specimens with narrow and wide columns showed pinching phenomena; but one was associated with bar slip caused by high bond stresses along beam reinforcement. However, the second was associated with shift of the beam nuetral axis caused by the existence of slab. The columns were designed to develop flexural strength 40 percent stronger than that of the beams.

Concluding Remarks by T.OKADA (Univ. of Tokyo, Japan)
At the end of this special theme session, I as a member of the coordinators would like to thank the chairpersons, the Introductory and the State-of-the-Art reporters, the authors of papers and participants who have made this session a success. I do believe that development of technology such as that discussed in this session starts from deep observation of real behavior. And after innovative analysis with both theoretical and empirical approaches, these can be implemented to design of structures. I think that contributions of this session are successful in the observation and the analysis stages. I hope that results of this session will be extended to develop researches toward the stage of implementation. It would be a pleasure for us to meet you again at the next WCEE and to find technological development in this field.

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