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DEFORMABILITY ESTIMATION OF REINFORCED CONCRETE COLUMNS AT LIMIT STAGE OF GRAVITY LOAD COLLAPSE

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To assess the lateral limit at which reinforced concrete columns may lose their ability to carry on applied vertical loads is of importance, especially when dealing with existing structures, in terms of human lives safety and buildings strengthening, if any. Actually, many influencing parameters, which are still under investigations, have an effect on columns' behavior beyond a certain level of deformability and assessment of the previously mentioned limit becomes complex.
A simple model based on shear friction along an assumed critical crack is considered to estimate roughly the lateral drift at column's ultimate stage. Results of experiments on reinforced concrete columns tested beyond their shear failure till full collapse under axial loads are used as basis to reach the proposed estimation.

Key words: *reinforced concrete columns, lateral deformability, collapse, lateral loading type, axial load*

1. INTRODUCTION

The assessment and evolution of the strength and stiffness of existing or newly designed critical reinforced concrete structures and the evaluation of the performance of such structures under various loadings, especially at very advanced stage of damage, strongly requires a well understanding of the phenomena affecting the behavior of these structures and all their individual components[1]. Basic procedures and simplifications are commonly used in order to separate influencing parameters before the development of advanced analytical methods capable of representing the behavior of the structure under all possible loading conditions and dealing with all aspects of the problem.

The seismic performance of reinforced concrete columns beyond their shear failure is investigated in term of maximum lateral deformability. A simple model is assumed at limit stage and when combined to experimental results, it allowed a rough evaluation of the lateral deformability that columns may reach under axial and lateral loadings. This work is intended to be a trial and also an introduction to future studies.

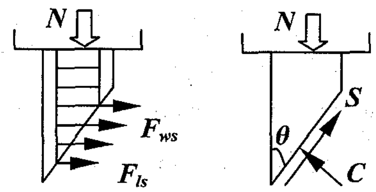

Fig.1 Assumed equilibrium forces for shear model

2. ASSUMED ANALYTICAL MODEL

The analytical method for assessing the lateral drift is based on the shear-friction model[2]. It assumes a diagonal failure plane along the column section height. The column would fail after shear failure and reach collapse when it would no more sustain the applied axial load. This stage is considered as an ultimate limit and it is attained

Table 1 Characteristics of tested specimens

Specimen	b (mm)	D (mm)	a (mm)	S_t (mm)	ρ_{ls} (%)	ρ_{ws} (%)	f_{ly} (Mpa)	f_{wy} (Mpa)	F_c (Mpa)	N (kN)	Lat. Load
Test 2001											
C1	300	300	450	160	1.693	0.083	340	587	13.5	364.5	type-1
C4	300	300	450	75	1.693	0.284	340	384	13.5	364.5	type-1
C6	300	300	450	75	1.693	0.284	340	384	13.5	V	type-1
C8	300	300	450	75	1.693	0.284	340	384	18.0	486.0	type-1
C10	300	300	450	75	1.693	0.284	340	384	18.0	V	type-1
C12	300	300	450	75	1.693	0.284	340	384	18.0	324.0	type-1
Test 2002											
C1	300	300	300	50	1.693	0.43	447	398	27.7	540	type-2
C16	300	300	300	50	1.693	0.43	447	398	26.1	540	type-3
C11	300	300	450	150	2.258	0.14	447	398	28.15	540	type-2
C12	300	300	450	150	2.258	0.14	447	398	28.15	540	type-3
C13	300	300	450	50	2.258	0.43	447	398	26.1	540	type-2
C14	300	300	450	50	2.258	0.43	447	398	26.1	540	type-3
C15	300	300	450	50	2.258	0.85	447	398	26.1	540	type-3
Literature: tests carried out by Sezen and Moehle, 2000											
2CLD12	457.2	457.2	1473.2	304.8	2.50	0.17	441.3	468.8	21.1	667.2	-
2CHD12	457.2	457.2	1473.2	304.8	2.50	0.17	441.3	468.8	21.1	2668.9	-
2CVD12	457.2	457.2	1473.2	304.8	2.50	0.17	441.3	468.8	20.9	V	-
2CLD12M	457.2	457.2	1473.2	304.8	2.50	0.17	441.3	468.8	21.8	667.2	-
Literature: tests carried out by Lynn and Moehle, 1996											
3CLH18	457.2	457.2	1473.2	457.2	3.00	0.10	330.9	399.9	25.6	502.6	-
3SLH18	457.2	457.2	1473.2	457.2	3.00	0.10	330.9	399.9	25.6	502.6	-
2CLH18	457.2	457.2	1473.2	457.2	2.00	0.10	330.9	399.9	33.1	502.6	-
2SLH18	457.2	457.2	1473.2	457.2	2.00	0.10	330.9	399.9	33.1	502.6	-
2CMH18	457.2	457.2	1473.2	457.2	2.00	0.10	330.9	399.9	25.7	1512.4	-
3CMH18	457.2	457.2	1473.2	457.2	3.00	0.10	330.9	399.9	27.6	1512.4	-
3CMD12	457.2	457.2	1473.2	304.8	3.00	0.17	330.9	399.9	27.6	1512.4	-
3SMD12	457.2	457.2	1473.2	304.8	3.00	0.17	330.9	399.9	25.7	1512.4	-

Notation: b= section's width, D= section's height, a= shear span (half of column height), S_t= stirrups spacing, ρ_{ls}= total longitudinal reinforcement ratio, ρ_{ws}= transverse reinforcement ratio, f_{ly}= longitudinal reinforcement yield strength, f_{wy}= transverse reinforcement yield strength, F_c= concrete strength, N= axial load, V= varying axial load, type-1 & type-2= simulation of far field earthquake, type-3= simulation of near field earthquake.

when the resulting sliding force S (**Fig.1**) along the failure plane reaches the plane tangent component of the compression force C by mean of friction μ (Eq.1). The resulting forces, C and S, respectively normal and tangent to the inclined plane are the result of the applied axial load N, the action of the force F_{ws} developed by the transverse reinforcement due to tension and the force F_{ls} developed by the longitudinal reinforcement due to dowel effect. The vertical action of the longitudinal reinforcement is disregarded due to buckling of steel bars. The column's shear strength at the moment of collapse is considered negligible. The assumption on the value of the inclined plane angle θ has not a negligible effect on the aimed results and very complex to formulate as it can be presented later on. Assessment of the friction along the presumed failure plane is a function of the parameters mentioned previously and expressed by

$$S = \mu C \tag{1}$$

where

$$C = N \sin\theta + (F_{ls} + F_{ws}) \cos\theta \tag{2}$$

$$S = N \cos\theta - (F_{ls} + F_{ws}) \sin\theta \tag{3}$$

By introducing the concrete strength F_c, using steel ratios, ρ_{ls} and ρ_{ws}, and steel yield strengths, f_{ly} and

f_{wy}, respectively, for longitudinal and transverse reinforcements, equation (1) is written as follows.

$$\mu = \frac{(1-K_1)\tan\theta}{K_1 + \tan^2\theta} \quad \text{...............(4)}$$

where

$$K_1 = \frac{\rho_{ws} f_{wy}}{\eta F_c} + \alpha \frac{\rho_{ls} f_{ly}}{\eta F_c} \qquad (5)$$

α is a factor counting for the dowel action, and
η is the axial load ratio

Even though equation (4) does not include any variable concerning the lateral displacement at which columns may experience collapse, the formulation will be used counting on some experimental data collected for the purpose of evaluating the ultimate lateral deformability at which columns may lose their ability to sustain vertical loads. Actually, the friction factor is regarded as a variable affected by the lateral deformability as well as some other parameters.

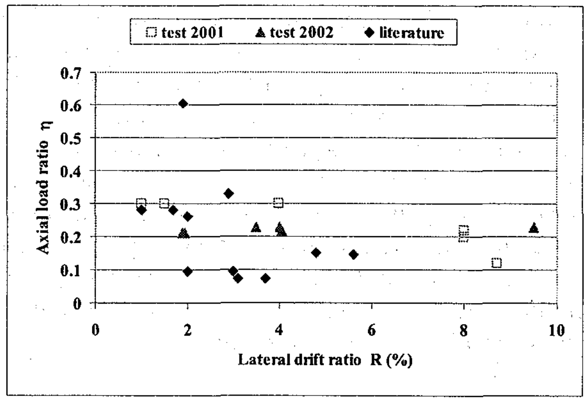

Fig.2 Performance of tested columns

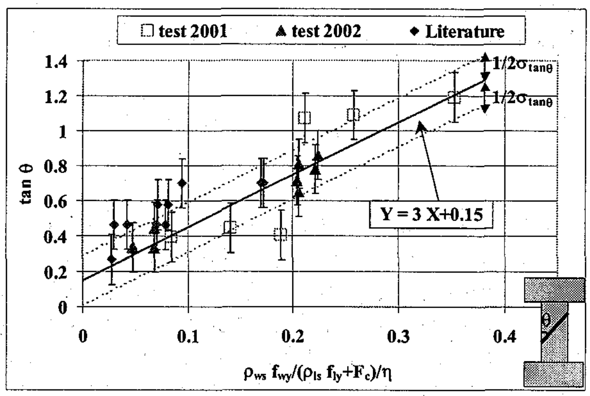

Fig.3 Variation of failure plane inclination

3. EXPERIMENTAL DATA

While some experimental data were the fruit of laboratory tests on specimens in which the author was involved in 2001[3] and 2002[4], some other data were cited by Moehle[2], as summarized in **Table 1**. Generally, tests on reinforced concrete columns are concluded soon after reaching shear failure, though tested columns still possess a certain residual capacity to sustain applied axial loads. Data gathered from literature[1] were selected among many others and contains only those of specimens, which went beyond shear failure till collapsed under vertical loads. Tested specimens represent common first story columns found in medium to high-rise buildings.

All tests were conducted on scaled square-section columns, which were subjected to axial and lateral loadings. Columns experienced laterally, anti-symmetric double curvature bending, where loading paths were controlled by displacement. Axial loads were, in general cases, constant and varying in few other cases. Different parameters were investigated. When lateral loading path was unchanged, three parameters were considered: concrete strength, shear reinforcement ratio and axial loading[3]. When axial load was unchanged (constant load), three parameters were selected: shear span ratio, shear reinforcement ratio and lateral loading[4].

The tests showed that cracking pattern is a very important parameter that influenced very much behavior of specimens, shear degradation, stiffness changes and affected the final stage at collapse[3, 4].

The performance of tested columns varied according to their geometric characteristics and mechanical properties, loading intensity and loading type. **Fig.2** illustrates the spread results, in term of applied axial load and maximum attained lateral drift. Also, Failure plane inclination angles that were observed on tested specimens were disparate and difficult to relate trough an adequate formulation like shown on **Fig.3**, which is considered as an acceptable fit to experimental data. Data calculated average and its standard deviation are, respectively, $\overline{\tan\theta}=0.753$ and $\sigma_{\tan\theta}=0.282$. The figure relates the observed failure plane inclination to some main parameters. It is worth to note here that inclination angles were assessed subjectively according to the judgment of the observer during testing and might include some unavoidable errors.

As to friction variation, considering or neglecting the dowel action of longitudinal reinforcement resulted in differences as to required friction values at columns' ultimate stage. On the basis of the observed inclination angles on tested specimens, the number of stirrups crossing the main cracks and the formulation given in equation (4), the friction, after some trials, was expressed relatively well when dowel action was considered and by combination of different parameters rather than by a single parameter, to name the lateral drift ratio (R). The friction, when dowel action was considered is shown in **Fig.4** and expressed by

$$\mu = 0.5 \left(k\, R\right)^{-0.36} \tag{6}$$

where

R expresses the lateral drift ratio in %, and

$$k = \frac{\rho_{ws}\, f_{wy}}{\eta\, F_c} \tag{7}$$

The formulation in equation (6) intends to be compared to the formulation given by equation (4) after expressing equation (5) in a simple way by means of experimental data. Actually, it was found that equation (5) might be expressed in terms of transverse reinforcement ratio, concrete strength and axial load ratio, as illustrated in **Fig.5** For the range of specimens mentioned herein, equation (5) is expressed as follows.

$$K_1 = 0.03 + 1.33\, k \tag{8}$$

4. LATERAL DRIFT EVALUATION

As it was mentioned previously, inclination of failure plane varied from one specimen to another specimen and was found hard to predict due to implication of different parameters and variables that influences beginning and progress of cracks. Although a trial to formulate the mentioned inclination conveniently is suggested, the spread data is still of concern. This fact drives the analysis to assume a value that might induce an optimum friction as higher as possible like given in the following.

For actual cases, the parameter K_1 is always positive and when considered as constant, the friction at collapse, computed from equation (4), would reach its maximum value when $\tan\theta = \sqrt{K_1}$, then

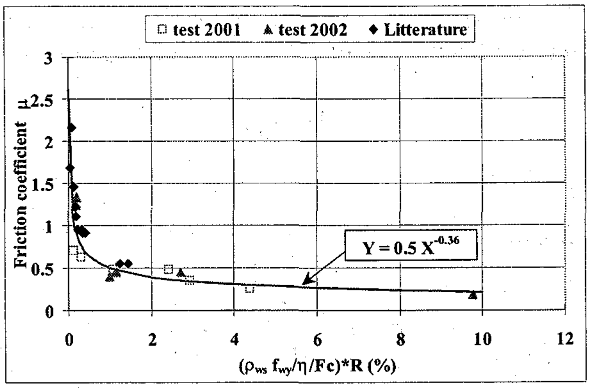

Fig.4 Friction variation from experimental data

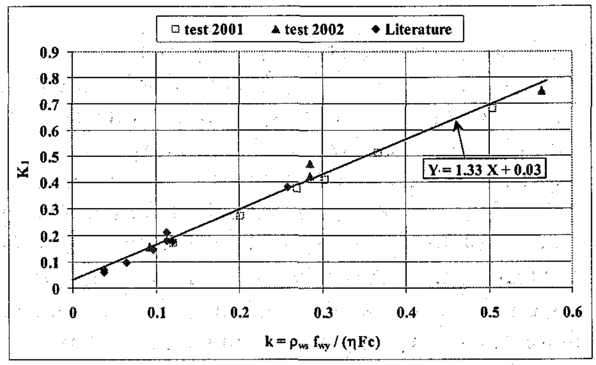

Fig.5 Resulted variation of K_1 factor

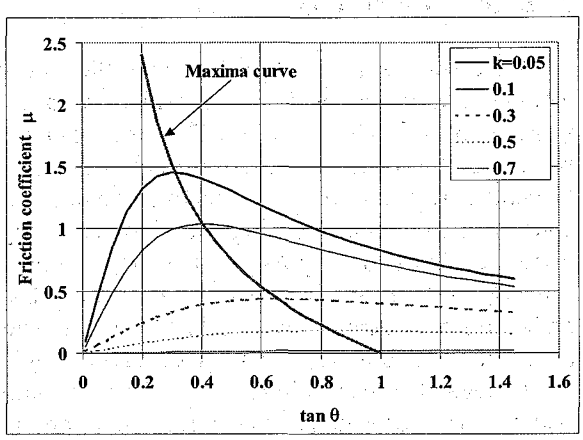

Fig.6 Variation of friction and maxima curve

$$\mu_{\max} = \frac{1 - K_1}{2\sqrt{K_1}} \tag{9}$$

The variation of maximum friction, according to the inclination angle and for different values of the parameter k, when equation (8) is integrated, is illustrated in **Fig.6**. When only maximums are considered, the line passing through all peaks (maxima curve) is then considered as an upper limit that cannot be overflowed. However, this line is not taken into account when the friction coefficient has negative values.

Finally, by assuming that the friction assessed by equation (6) is equal to the friction assessed by equation (9), the lateral drift R at limit stage can be written in terms of the parameter k as given in equation (10).

$$\boldsymbol{R}\,(\%) = \frac{1}{\boldsymbol{k}}\left(\frac{0.97 - 1.33\boldsymbol{k}}{\sqrt{0.03 + 1.33\boldsymbol{k}}}\right)^{1/0.36} \qquad (10)$$

Equation (10) is illustrated in **Fig.7**, which includes also the experimental results. The obtained curve forms a lower boundary for the majority of experimental data involved in this study. While the curve surrounds all experimental data for specimens considered relatively slender with large spacing of stirrups (data gathered from literature), it does not enclose some experimental data for specimens considered relatively short with closer spacing of stirrups. Although this deficiency, which might be due to omitting to take into account some other influencing parameters, like the effect of lateral loading type in the model, or the effect of the shear span ratio, roughly, the curve can be considered as a limit of the lateral drift that columns might assure under combined axial and lateral loadings.

According to the formulation, it suggests that lateral drift increases when parameter k increases. While this fact is acceptable for a certain range of parameter k, the proposed formulation would be unsatisfactory after a certain value of the parameter k where the assessed value would be far higher than the actual one, as it can be seen for one of the tested specimens (k ≈ 0.5) in **Fig.7**. To prevent such case, an upper limit should be fixed from a reasonable level at which no reinforced concrete columns with insufficient stirrups would be expected to perform beyond it. For instance, by referring to the present experimental data, a maximum lateral drift ratio of 10% might be an acceptable limit when the parameter k reach a value higher than 0.4. However, it is worth to note here that this formulation is based on the assumptions mentioned in Chapter 2, where behavior of well-confined columns is not of concern.

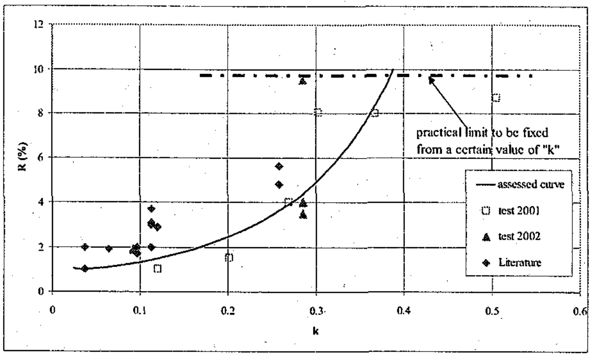

Fig.7 Proposed curve of lateral drift

5. CONCLUSIONS

A simple shear-friction model has been used to assess the limit lateral drift of reinforced concrete columns, which are insufficiently outfitted transversally with stirrups. The analysis took as basis some experimental data carried by the authors and some others from literature. In order to ensure, statistically, relatively wide range-samples, various geometric and material characteristics were selected. Also, different loading types, laterally and axially, were applied.

Combining analytical and empirical formulations lead to a rough evaluation of the lateral drift at a stage where columns would not be any more able to sustain applied vertical loads. The lateral drift ratio is evaluated directly as a function of the applied axial load, the concrete strength and the transverse steel content. The action of longitudinal reinforcements is not considered because of a probable buckling of steel bars, however, their dowel action is, indirectly, taken into account. While for some ranges of axial load, concrete strength and transverse steel, the proposed formulation gives satisfaction, for other ranges of the same parameters, the formulation might need to include some other parameters to reach a better conclusion. According to experimental data, probably by introducing some parameters like the slenderness of columns or loading type the formulation would be improved significantly.

While this formulation is regarded as a result of a limited number of tested specimens, it cannot be intended as a general to all reinforced concrete columns. More data and more understanding on the behavior of such columns are needed to reach a better assessment of the deformability of reinforced concrete columns at limit stages.

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