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Design of Slab-Column Frames

Conception de cadres dalles-colonnes

Entwurf von Stahlbeton-Plattenträger-Strukturen

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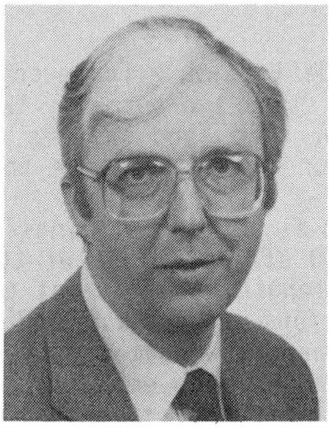

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SUMMARY
In the last 10 years a significant number of slab-column frame structures have been designed and constructed in the Washington D.C. area. Flat plate construction has been very popular for many years in Washington because of the stringent building height limitations caused by a desire not to overshadow the nation's Capitol building. The use of slab-column frames allows a 10-story office building with a total height limitation of approximately 32 m. Elimination of exterior masonry walls and interior masonry partitions led designers to look for other methods of resisting wind loads. Hence, the development of the slab-column frame.

RÉSUMÉ
Dans les 10 dernières années, un nombre important de structures en cadres dalles colonnes ont été dimensionnées puis réalisées dans les environs de Washington DC, USA. La construction de bâtiments relativement plats a été très répandue depuis de nombreuses années du fait des restrictions imprératives sur la hauteur des édifices en vue d'éviter qu'ils ne dominent le Capitole lui-même. C'est ainsi que l'utilisation des structures en cadre dalles-colonnes permet de construire des immeubles de bureaux de 10 étages, dont la hauteur n'excéde pas 32.00 m. L'élimination des parois extérieures et des galandages en maçonnerie a nécessité la recherche d'autres méthodes pour contreventer les bâtiments, d'où le développement de l'étude des cadres dalles-colonnes.

ZUSAMMENFASSUNG
In den letzten 10 Jahren wurden in Washington, DC viele Flachdecken entworfen und gebaut. Flachdecken waren in Washington auf Grund der Höhenbeschränkung für Gebäude, um das Kapitol nicht zu beschatten, für viele Jahre weit verbreitet. Die Verwendung dieser Bauweise machte es möglich, ein zehngeschossiges Bürogebäude mit einer Gesamthöhe von etwa 32 Metern zu errichten. Durch das Weglassen von Aussen- und Innentragwänden waren die Entwerfer gezwungen, die Rahmenwirkung mit den Stützen heranzuziehen.

1. INTRODUCTION

In the last 10 years a significant number of slab-column frame structures have been designed and constructed in the Washington, D.C. area. Flat plate construction has been very popular for many years in Washington because of the stringent building height limitations caused by a desire to not overshadow the nation's capitol building. The use of slab column frames allows for the construction of a 10 story office building with a total height limitation of approximately 105'. Elimination of substantial exterior masonry walls and interior masonry partitions caused designers to look for other methods of resisting wind loads. Hence, the development of the slab-column frame.

A slab-column frame is by definition a frame composed of slabs and columns in lieu of beams and columns. The slabs are normally two-way systems such as flat plates, flat slabs, waffle slabs or slab band systems. The purpose of using these types of systems is basically economy and a desire to avoid the use of shear walls.

In many building types such as office buildings, the use of shear walls causes many impediments to planning and efficient use of the floor space. If the core areas provide enough shear wall length, this is not a problem but often that is not the case and supplemental locations must be used. The economy of cast in place concrete construction is greatly enhanced when the only formwork which is required is flat deck forming and column forming with emphasis on keeping columns the same size as much as possible. The formwork cost increases significantly with the addition of concrete beams and shear walls. As a result it has been proven to be quite economical to use slab-column frames as the systems for buildings of moderate height (up to 12-14 levels) and in areas of low seismic risk.

This type of system works very well in hotel room towers as it does for office buildings. Some of the same reasons such as a desire to maintain a minimal floor to floor height make the slab-column frame an ideal solution for hotel towers up to approximately 12 floors. The normal gravity load system is a post tensioned flat plate with a thickness of 6 1/2" or 7". When the moment transfer requirements in the slab at the lower floors become a very large magnitude it is necessary to go to another lateral load resisting system. Normally this would occur at around 12 levels and would require the use of a shear wall system for heights greater than that.

2. DESIGN BASIS

2.1 Gravity Load Design

The gravity load design of slab-column frames is commonly accomplished utilizing the equivalent frame method or some PC based software. PCA's ADOSS is the program commonly used for mild reinforced systems and our office uses the post tensioning program developed and marketed by Structural Data, Inc. for the design of post tensioned floor systems. The normal procedure is to design the floor system for gravity loads and then to check it for the combination of lateral loads and gravity loads. In most cases this will require the addition of small amounts of bonded reinforcing which should be concentrated in the area over the column and immediately adjacent to each side of the column.

2.1 Lateral Load Design

Although the gravity load design of slab-column frames is similar to other systems, the design for lateral loads places some added constraints on the designer. The slab-column frame is a very flexible system so that drift or lateral deflections are of primary importance. The calculation of lateral deflections are greatly affected by the assumptions.

Our office practice is as follows:

1. All buildings are designed for strength as per the appropriate building code. This is normally the Building Officials & Code Administrators (BOCA Code) in the Washington, D.C. area. The design wind speed is generally 80 or 90 mph.

2. The normal criteria which we use for wind drift is h/500 or h/400 where h is the height of the building under design. THIS IS NOT A CODE REQUIREMENT.

3. The wind load used for drift calculations is related to the highest recorded wind speed in the area. In the Washington area we would generally use 60 mph for our drift calculations. Our theory is that in most buildings discomfort due to movement should be avoided for normal or probable loadings. These are not necessarily the same as CODE loadings. It is easy to see that the calculated drift is less than half if 60 mph is used in lieu of 90 mph.

4. Reduced stiffnesses are used for the slab-column frames. The slab stiffness is based on 35% of the total bay width and the columns on 100% of their gross stiffness.

5. The lateral load analysis is accomplished with either ETABS, a program developed at the University of California and marketed by Computers & Structures, Inc., or one of several plane frame programs utilizing various modeling techniques.

A similar procedure is used for consideration of Zone 1 earthquake loadings. We are presently doing our first project in a Zone 2 area utilizing a slab-band system and the provisions of Chapter 21 in ACI 318-89 and the 1985 Uniform Building Code (UBC) allowing the use of two-way slab systems in zones of moderate seismic risk.

2.3 Column Design

Utilizing the information from the gravity load design of the slab system and the lateral load analysis of the slab-column frame our office uses a simplified moment magnification approach as proposed by MacGregor for medium height structures. A more exact method would be required for tall structures. This approach is valid when the stability index Q is between 0.0475 and 0.20. Q is defined as $\Sigma P_u \Delta_1 / H_u h_u$. P_u is the factored load of the entire story, Δ_1 is the first order story deflection. H_u is the total lateral load for this story and the stories above, and h_u is the unsupported clear height of the column.

The method is described in STABILITY ANALYSIS AND DESIGN OF CONCRETE FRAMES, James G. MacGregor and Sven E. Hage, ASCE Structural Journal, October 1977.

The structural magnified moments which will normally be significantly less than those utilizing the CODE equations are then combined as appropriate with the factored loads and moments from the gravity load analysis.

2.4 Shear Design

Typical two-way slab systems utilize a "shear cap" to increase the shear capacity of the slab system in the critical area surrounding the column and to allow the subsequent use of a thinner slab controlled by flexure and deflection considerations rather than shear.

The 1983 ACI 318 basically did not allow for the contribution of a column capital or bracket which lay outside a 45° tapered wedge. Consequently if the horizontal extension of a "shear cap" was greater than the vertical depth of the "shear cap", the added horizontal concrete was of no value in calculating the shear capacity of the slab system. This meant that if a 4'-0" square "shear cap" that was 6" deeper than the slab was used with a 24" square column only the first 6" projection contributed to the shear capacity of the system and the second 6" projection did not affect the capacity. Obviously, this was not correct and a subsequent change in the 1989 CODE gives a basis for utilizing that capacity.

The 1989 ACI 318 requirement requires a gradual decrease in v_c from $4\sqrt{f'_c}$ to $2\sqrt{f'_c}$ as the ratio of the critical perimeter b_o, to the critical depth, d, increases. The requirement will mean that at the edge of a 4'-0" square "shear cap" with an 8" slab such as might be used on an office building, the allowable v_c would be $3.32\sqrt{f'_c}$ in lieu of $4\sqrt{f'_c}$ as might have been assumed for an interior column.

ACI-ASCE Committee 352 also has a recommendation in their "Tentative Recommendation for Design of Slab Column Connections in Monolithic Reinforced Concrete Structures". Their requirement is based on just two adjustments. If the ratio of the maximum perimeter of the slab critical section to slab effective depth is less than 20, the allowable $v_c = 4\sqrt{f'_c}$. If the value is between 20 and 40, v_c is modified to $3\sqrt{f'_c}$ and if it is over 40 v_c becomes $2\sqrt{f'_c}$. Our sample from above would have a ratio of 4(48+d) over d. If we assume that d is 7 than 212/7 = 30.3 so $v_c = 3\sqrt{f'_c}$. Obviously, all of these values in any case would need to be adjusted as appropriate for other factors such as the aspect ratio of the column.

3. TEST PROGRAM

A small test program involving eight specimens was accomplished at the University of California at Berkeley in 1988 and '89 under the direction of Dr. Jack Moehle. The purpose of the program was to attempt to determine the effectiveness of a "shear cap" in increasing the shear capacity of a slab-column joint. The results of the testing were the confirmation of the allowable shear values indicated in both ACI-318-89 and the ACI-ASCE 352 report. The values are apparently conservative, but the failure mode is not as anticipated. The "shear caps" do not act as an extension of the column area but are much more flexible and actually tend to fall off the bottom of the slab. As previously noted, the allowables as proposed are appropriate.

4. DETAILING

Both the ACI-ASCE Committee 352 recommendations and the 1989 ACI 318 CODE require that some minimum amount of positive (bottom) reinforcing be continuous through the joint.

In slab-column frames there is often a requirement at the lower floors for continuous bottom reinforcing through the joint for strength considerations.

ACI-ASCE 352 and ACI 318-89 require some continuous bottom reinforcing in two-way slab systems at the supports regardless of the lateral load resisting system utilized for the building.

5. CONCLUSIONS

Slab-column frames have proved to be a viable, economical solution for many building projects in the Washington, D.C. area as well as other parts of the United States. In the last 3 or 4 years our office has designed at least 6 million square feet of construction utilizing this system. The system allows construction of large bay (30' x 30') or medium bay (20' x 20') office buildings with minimal floor to floor heights, since the effective structural depth is often no more than 8".

Although the system has limitations particularly for heavy seismic areas, it is a good solution for buildings in a concrete town like Washington, D.C. or any area of low-seismic risk.