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Strength and ductility of reinforced concrete columns with interlocking spirals

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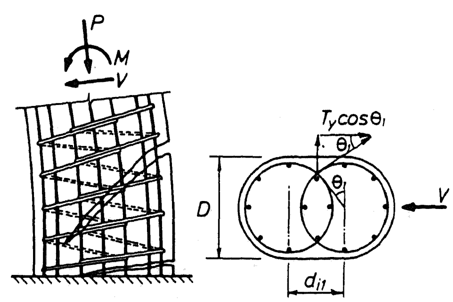
ABSTRACT: Design methods to ensure ductile behaviour of reinforced concrete columns using interlocking spirals as shear and lateral confining reinforcement are proposed. The adequacy of the proposed methods were verified by conducting cyclic horizontal loading tests on three columns with interlocking spirals and, for comparison, one column with rectangular hoops and cross ties. Based on the crack patterns of concrete and strain behaviour in the longitudinal reinforcement observed in those tests, a combined beam arch-action model was proposed to predict the flexural strength of those columns. This model can predict the column flexural strength more adequately than ordinary beam action theory when the interlock of spirals is weak. A shear deformation model was also proposed to interpret the deformation mechanism which actually occurred in the test columns with interlocking spirals.

1 INTRODUCTION

A column section, such as shown in Fig. 1, has been introduced by the California Department of Transportation (CALTRANS 1983) for bridge structures designed for seismic loading. This section with interlocking spirals is preferred by CALTRANS to sections with rectangular hoops and cross ties. This is firstly because the amount of transverse reinforcement required to confine the core concrete, in order to provide adequate ductility in potential plastic hinge region of columns, may be considerably reduced as a result of the higher confining efficiency of spirals compared with rectangular hoops and cross ties. The second reason is that the fabrication of reinforcement is easier when using interlocking spirals instead of overlapping rectangular hoops or rectangular hoops with cross ties. However, it appears that no past experimental or theoretical study, except for that by the authors (Tanaka & Park 1990), has been conducted to evaluate the effectiveness of interlocking spirals as shear and lateral confining reinforcement in columns.

In the previous study conducted by the authors (Tanaka & Park 1990) a series of methods to evaulate the effectiveness of interlocking spirals as shear and lateral confining reinforcement were proposed. The adequacy of those proposed methods was assessed by carrying out cyclic horizontal loading tests on three columns with interlocking spirals and, for comparison, one column with rectangular hoops and cross ties. In this study, reinforcing details to ensure adequate ductility of column using interlocking spirals are first reviewed. Secondly, for the case when the interlock of spirals is weak, a combined beam-arch action model to predict flexural strength of columns with interlocking spirals is proposed. Thirdly, a model to evaluate shear deformation of a column with interlocking spirals is proposed.



(a) Longitudinal Section (b) Transverse Section

Fig.1 Reinforcing details for a column with interlocking spirals and longitudinal bars.

2 REINFORCING DETAILS TO ENSURE DUCTILE BEHAVIOUR OF CONCRETE COLUMNS USING INTERLOCKING SPIRALS

2.1 *Secure interlock of spirals*

When horizontal seismic force is applied to the column in the direction y as shown in Fig. 2, the

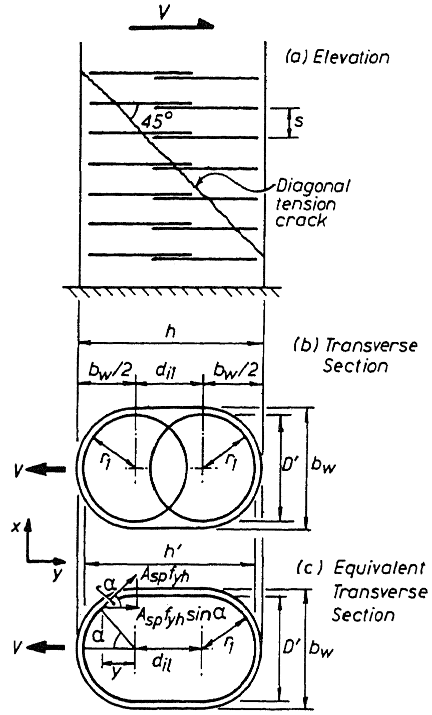


Fig.2 Shear carried by interlocking spirals.

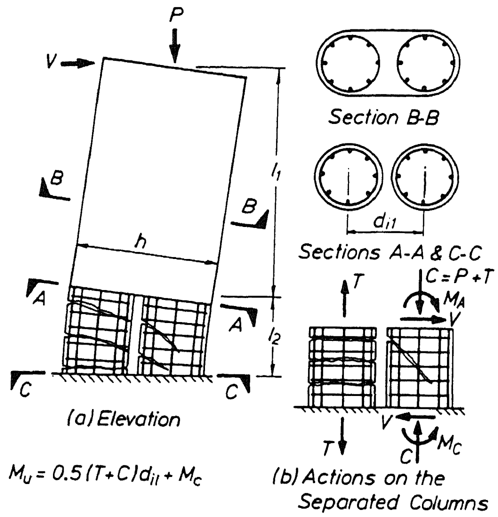


Fig.3 Combined beam-arch action model for the case when the interlock of spiral is weak.

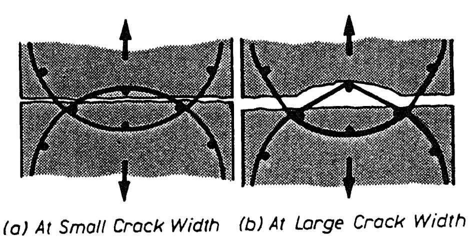


Fig.4 Deformation of interlocking spirals due to diagonal shear cracking.

effectiveness of the interlocking spirals as shear reinforcement will depend on how securely the spirals are interlocked. If the interlock of the spirals is weak, vertical splitting cracks parallel with the column axis may be formed in the region of spiral interlock under severe seismic loading and could separate the column into two parts after spalling of cover concrete, as modelled in Fig.3.

In the study by the authors (Tanaka & Park 1990) it is recommended that the distance between centres of the interlocking spirals, d_{il}, should not be greater than 1.2 times the spiral radius in order to provide sufficient area of interlock for adequate shear transfer. It must be noted that the component of spiral bar yield force, Ty $\cos\theta_1$, at mid-depth of the column section acting in the direction of the applied shear force in Fig. 1, is reduced as the distance d_{il} increases. The magnitude of Ty $\cos\theta_1$, can be more than 80 % of the yield force of the spiral bar, when d_{il} is less than 1.2 times the spiral radius.

It was also recommended that at least four longi-tudinal bars shoud be provided inside the interlocking area of the spirals in order to ensure adequate shear transfer between the interlocking spirals, based on the assumed shear transfer mechanism shown in Fig. 4.

2.2 *Interlocking spirals as shear reinforcement*

For a circular column with single spiral, in the study by Priestley and Park (1984) it was shown that the shear carried by the spirals can be estimated by summing the components in the direction of the applied shear force of the spiral bar forces intercepted by a 45° crack. Assuming that all parts of the spiral bars intercepted by the 45° crack yield in tension, the shear carried by a single spiral is

$$V_s = \frac{\pi}{4}(2A_{sp}f_{yh})\frac{D'}{s} \qquad (1)$$

where A_{sp} is the area of spiral bar section, f_{yh} is the yield strength of the spiral steel, D' is the diameter of the circular array of longitudinal reinforcement and s is the centre to centre spacing of spirals.

When in the case of a column with two interlocking spirals the horizontal shear force is applied in the direction y as shown in Fig.2, the shear carried by the interlocking spirals cannot be estimated simply as twice that of a single spiral, unless the spirals are perfectly interlocked. When the spirals are interlocked in accordance with the recommendations in the previous section, an adequate estimate of the shear carried by the interlocking spirals can be obtained by assuming that only the peripheral spiral bars outside the area of interlock are effective to carry the horizontal shear. Hence the effectiveness of those peripheral spiral bars is almost equivalent to that of the transverse reinforcement shown in Fig. 2(c), and the shear carried can be shown to be

$$V_s = \frac{\pi}{4}(2A_{sp}f_{yh})\frac{D'}{s} + 2A_{sp}f_{yh}\frac{d_{il}}{s} \qquad (2)$$

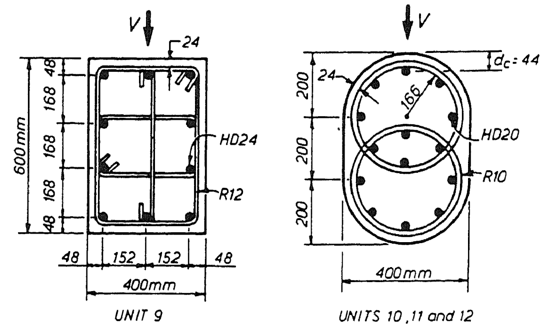


Fig.5 Column section details for Units 9 to 12.

2.3 *Tests on columns with interlocking spirals*

Cyclic horizontal loading tests were carried out on three columns with interlocking spirals and, for comparison, one rectangular column with rectangular hoops and cross ties. The column section dimensions were 400 mm by 600 mm and the ratio of shear span length to overall depth of column section was three. The details of column sections and the loading arrangements are shown in Figs. 5 and 6, respectively. For Units 10 to 12, the distance between centres of the interlocking spirals, d_{il}, was about 1.2 times the spiral radius, and four longitudinal bars were placed inside the area of interlock in accordance with the recommendations mentioned in Section 2.1. During the cyclic horizontal loading, a constant axial load of either 0.1, 0.3 or 0.5 f'_cA_g was also applied as shown in Table 1, where f'_c is the compressive cylinder strength of concrete and A_g is the gross area of column section.

Table 1 also lists the amount and spacing of transverse reinforcement provided in the potential plastic hinge region of each column. The amount of transverse reinforcement was determined to satisfy the provisions of the New Zealand concrete design code (1982) for both lateral confinement and shear, and was governed by the confinement requirement in the cases of Units 9, 11 and 12, and by the shear requirement in the case of Unit 10. The volume of interlocking spirals required for confinement by the code was estimated from the code equations for single spirals. The shear carried by the interlocking spirals was estimated assuming that the spirals were perfectly interlocked; that is, twice that of a single

Table 1 : Properties of the Column Units

Unit	$\frac{P}{f'_c A_g}$	f'_c (MPa)	Transverse Reinforcement in Plastic Hinge Regions				
			Bar Type and Diameter (mm)	s_h or s (mm)	f_{yh} (MPa)	ρ_s (%)	$\frac{\rho_s}{\rho_{scode}}$
9	0.1	26.9	R12	80	305	2.17	-
10	0.1	21.2	R10	80	308	1.08	1.09
11	0.3	29.7	R10	100	308	0.92	0.84
12	0.5	24.6	R10	75	308	1.15	0.97

Notes: The yield strength f_y of the HD24 longitudinal bars of Unit 9 was 432 MPa and of the HD20 longitudinal bars of Units 10, 11, and 12 was 485 MPa.
ρ_t was 1.88% for Unit 9 and 2.14% for Units 10, 11 and 12.
ρ_s = volumetric ratio of transverse reinforcement to concrete core. For Units 10 to 12, $\rho_s = 2\pi r_1 A_{sp}/(\pi r_1^2 s)$
ρ_{scode} = For Unit 10, ρ_s required to meet the New Zealand code provisions for shear; and for Units 11 and 12, ρ_s required to meet the New Zealand code provisions for confinement assuming $\phi = 1$ and $A_g/A_c = (r/r_1)^2$.

spiral calculated from Eq.1. If Eq. 2 is used, the estimate becomes 10% lower, when d_{il} is 1.2 times the spiral radius as in these test columns. The shear stress resisted by concrete in the potential plastic hinge regions, v_c, was zero, zero, $0.358\sqrt{f'_c}$ and $0.506\sqrt{f'_c}$ for Units 9, 10, 11 and 12, respectively, according to the New Zealand code (1982).

Horizontal load versus horizontal displacement hysteresis loops measured for all the column units are

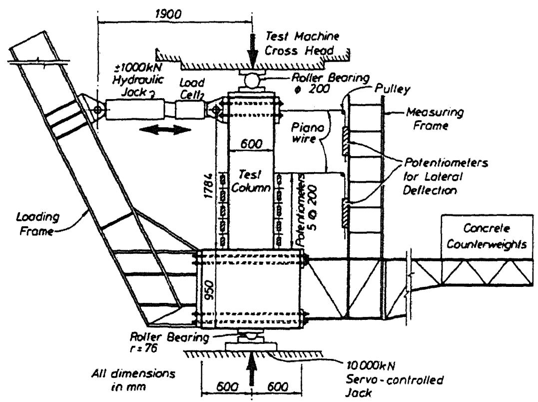


Fig.6 Loading arrangements for Units 9 to 12.

shown in Fig.7. The dashed lines in the figures show the theoretical ultimate horizontal load capacities V_{code}, including the P-Δ effect. V_{code} was calculated using the measured strength of steel and the ACI concrete compressive stress block (ACI 318-89), and assuming a strength reduction factor ø of unity. As can be seen from those figures, all the measured hysteresis loops indicated stable behaviour, good energy dissipation and limited reduction in strength up to the end of the test at displacement ductility factors μ of more than 8. Hence, as far as these tests are concerned, it may be concluded that the design method used was appropriate to provide adequate ductility for reinforced concrete columns with interlocking spirals.

3 A COMBINED BEAM-ARCH ACTION MODEL

When the interlock of spirals is weak due to large d_{il} or inadequate arrangement of longitudinal reinforcement to interlock spirals, the combined beam-arch action model shown in Fig. 3 can be used to estimate the flexural strength of the column. This model is appropriate for the case when the column axial load is small enough to make the neutral axis depth significantly less than one half of the overall depth of the column h.

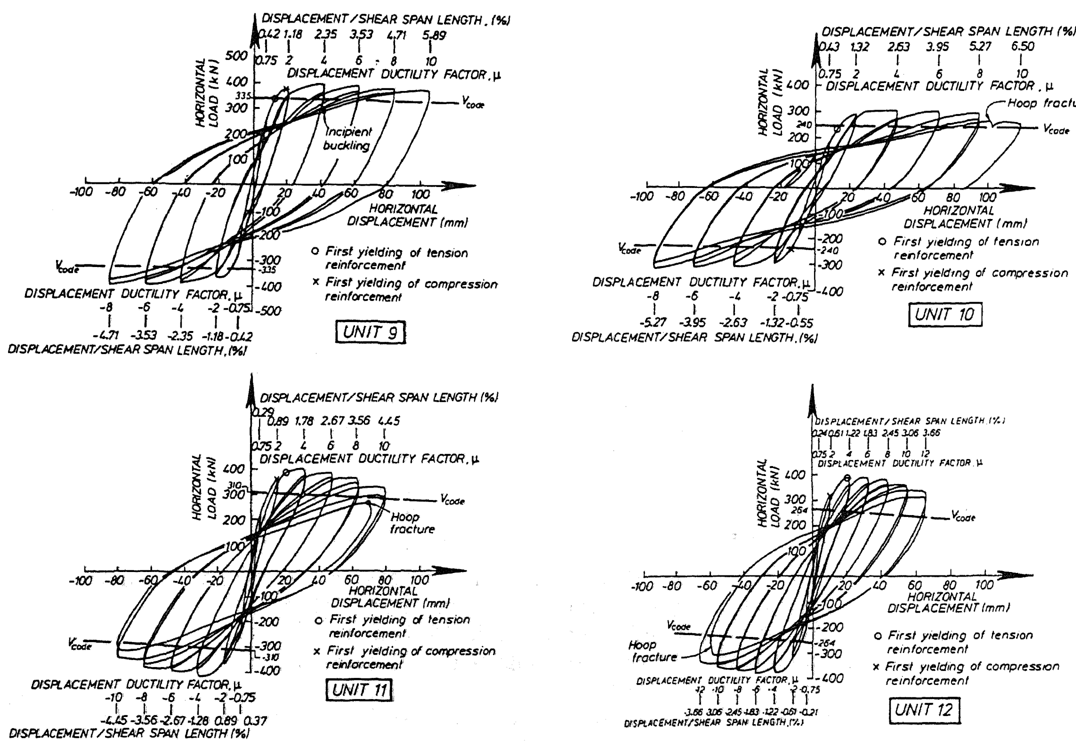


Fig.7 Measured horizontal load-horizontal displacement hysteresis loops for Units 9 to 12

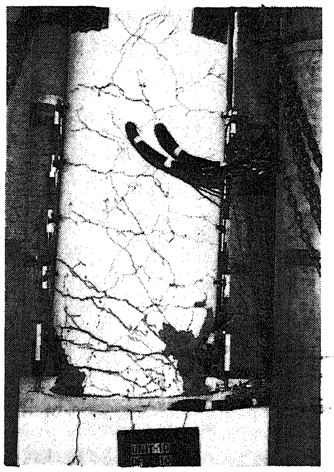


Fig.8 Photograph of Unit 10 at the end of test.

In the model the column is considered as two separate columns with single spirals. It is assmed that in the left hand column all bars, which have the same size and are evenly spaced, are yielding in tension and this separate column carries no bending and no shear. It is assumed that the resultant tension force T acts at the centroid of the left hand column section. It is assumed that the right hand column resists axial compression C = P + T, shear V and moment M_c, where P = axial load imposed on the total column. M_c can be calculated by conventional analysis assuming that plane section remains plane. The moment of resistance for the total column at the base is then

$$M = 0.5(T + C)d_{ll} + M_c \qquad (3)$$

The above combined beam-arch action model was applied to Units 10 to 12. For these three units the crack pattern observed was very similar to the model (see Fig. 8) and significant bond slip of longitudinal reinforcement was observed from an early stage of loading. The assumed column section is shown in Fig. 9. In the figure, the part of column section surrounded by solid lines is assumed to be subjected to the axial compression load C (=P+T) and bending moment. For this part of column section the moment of resistance M_c was found using the stress-strain models for unconfined and confined concrete proposed by Mander et al (Priestley & Park 1984) for the cover and core concrete, respectively. The compressive strain of cover concrete at spalling was assumed to be 0.6 %. As an example, the variation in the total moment of resistance given by Eq.3 for Unit 10 are plotted against the concrete compressive strain in the extreme compression fiber in Fig. 10. In this figure, the measured skeleton

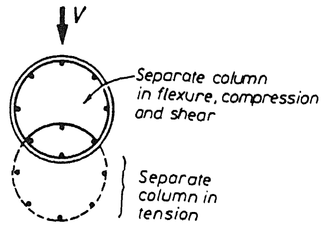


Fig.9 Assumed separate columns for combined beam-arch action models.

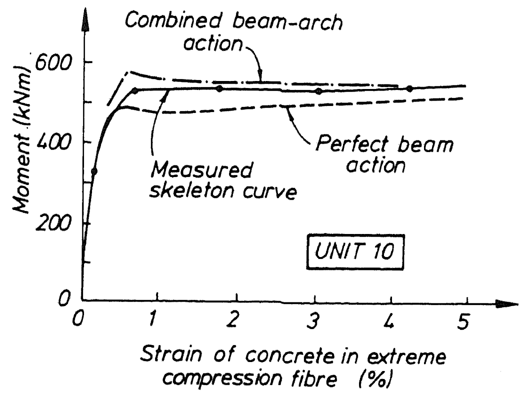


Fig.10 Comparison between theoretical and measured moment-concrete strain relations for Unit 10.

curve and the result calculated by assuming the perfect beam action are also indicated.

In case of the columns tested with interlocking spirals, the curves predicted by assuming the combined-beam arch action showed better fit with the measured skeleton curves than the curves obtained assuming perfect beam action. The combined beam-arch model can be seen analysed in more detail elsewhere (Tanaka & Park 1990).

4 SHEAR DEFORMATION

For columns tested the ratio of column deflection due to shear to total deflection reached 10 to 30 %. This ratio was almost constant from displacement ductility factor μ = 0.75 to the final stage of loading where μ was more than 8. Hence, it is suggested that for a column with an aspect ratio of 3, when the column deflection is calculated on the basis of the curvature distribution assuming that plane sections remain plane, the calculated column deflection should be increased by about 20 % to account for shear deformation.

It is suggested that, for columns with weak

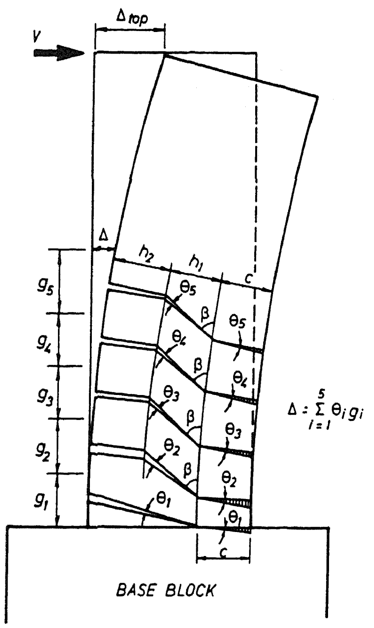


Fig.11 Assumed shear deformation model.

interlocking spirals or with comparatively small aspect ratio, the deflection can be estimated using the proposed shear deformation model shown in Fig. 11. The distance of the rotation centre from the extreme compression fibre denoted by c in the figure can be estimated as the neutral axis depth of the separated column subjected to $C = P + T$ in the combined beam-arch action theory. The ratio of h_1 to h_2 may be taken as unity and β may be assumed to be $45°$. It is not easy to rigorously determine the spread of the inelastically tensioned zone, but it may be taken as the plastic hinge length which is normally adopted for ordinary flexural deflection analysis. Using this model and the strains at the extreme tension fibres measured by the potentiometers, the column deflections for Units 9 to 12 were calculated. Those calculated deflections agreed well with the measured ones.

5 CONCLUSIONS

The main conclusions reached from this study were:

1. The amount of transverse reinforcement required for the confinement of the core concrete in the potential plastic hinge region of a column can be considerably reduced by using interlocking spirals instead of rectangular hoops and supplementary cross-ties.

2. The shear carried by interlocking spirals can be estimated by a proposed method which considers the equivalent transverse reinforcement.

3. When the interlocking of spirals is weak or the ratio of shear span length to section depth of column is small, the load carrying capacity of the column can be estimated by the combined beam-arch action theory proposed in this study.

4. It is recommended that to ensure the interlock of spirals, the distance between centres of interlocking spirals should not be greater than 1.2 times the spiral radius and at least four longitudinal bars should be provided inside the interlock area of the spirals.

5. The columns tested with interlocking spirals satisfied the above recommended requirements and the New Zealand code provisions for spirals. The behaviour of those columns under simulated severe earthquake loading was very satisfactory.

6. For the columns with an aspect ratio of 3, the shear deformation measured accounted for 10 to 30 % of the column deflection and this ratio was almost constant from displacement ductility factor $\mu = 0.75$ to the final stage of loading where μ of more than 8 was applied. Using the shear deformation model proposed here, the deformation mechanism that actually occurred in the tested columns was satisfactorily interpreted.

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