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STRUCTURAL BEHAVIOR OF R/C DEEP BEAM WITH HEADED LONGITUDINAL REINFORCEMENTS

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SUMMARY

This study is aimed to investigate whether the standard hook anchorage designed according to ACI 318-02 at the ends of the positive moment region can be replaced with mechanical anchorage using steel head and to estimate the shear behavior of deep beams. Eight deep beam specimens with headed reinforcements for mechanical anchorage and two general deep beam specimens with standard hook are designed. Main variables considered in the test are the anchorage type and shear span-to-overall height ratio. Two point static loads applied to the specimen and displacements are measured to collapse of the specimen.

From the study, it was found that the specimen with headed reinforcements as a mechanical anchorage showed better load resistance capacity when it was designed to satisfy the development length requirement of the ACI code. From this, it can be expected that the headed reinforcement has structural capacity compatible to the hooked bar when it is used as longitudinal reinforcements in RC deep beam design. In predicting shear strength of deep beam, Strut-and-Tie Model of Appendix A in ACI 318-02 was conservative and showed lowest standard deviation among several design methods.

INTRODUCTION

Reinforced concrete deep beam is defined that members with clear spans in equal or less than four times the overall member depth or regions of beams that are loaded on one face with concentrated loads within twice the member depth from the support and supported on the opposite face so that compression struts can be developed between the loads and supports.

In case of the building system composed of bearing wall and moment frame as upper and lower part, respectively, the load of upper part is transferred to column of another lane through the transfer girder. Therefore, the transfer girder is to be under high shear stress so that the depth of it is to be deeper. For this kind of beam, recently, the importance regarding the bonding of reinforcement and its anchor has been increased because high stress in accordance with the demand for the bulky and high-rise building is developed and transferred into concrete structure, which maximizes the concentrated stress at anchorage. In the reinforced concrete, the standard hook used in exterior beam-column joint is considered as an effective method for proper fabrication and construction to satisfy the requirement of design code that

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ductile tension failure instead of concrete failure should govern the over behavior of a member. However, standard hook tends to decrease the construction and economic efficiency due to the congestion of reinforcements in an anchorage region. Recently the study on alternative mechanical anchorage for the standard hook in exterior beam-column joint and deep beam has been presented[1,2].

This study is to investigate whether the standard hook anchorage designed according to ACI 318-02[3] at the ends of the positive moment region can be replaced with mechanical anchorage using steel head and to estimate the shear behavior of deep beams. Based on the results from the monotonic loading test of deep beam, the shear design procedures contained in the ACI 318-99 11.8[4], Strut-and-Tie model of ACI 318-02 and CSA A23.3-94[5] are evaluated.

EXPERIMENTAL PROGRAM

Test specimens

Ten specimens are planned according to experimental variables: anchorage type of longitudinal reinforcement, shear span-to-overall height a/h, vertical shear reinforcement ratios ρ_v and horizontal shear reinforcement ratios ρ_h. Table 1 and Fig. 1 show the detail of all specimens that have rectangular cross section with size of 160mm×600mm×2500mm.

Specimens are classified as two groups according to the anchorage type of longitudinal reinforcement. The first one (A) is the group designed to have longitudinal reinforcements with 90-degree hooks and the second group (M) is with mechanical anchorage. The steel head device shown in Fig. 2 is applied to the specimen as a mechanical device compatible to the hook anchorage. In the specimen details, vertical shear reinforcement is designed as closed stirrups type of 10mm deformed bars, while the horizontal shear reinforcement is straight type of 10mm deformed bars. All specimens are planned to have same concrete compressive of 40MPa.

Table 1 Detail of specimens

Specimen	a/h	Longitudinal Reinforcement		Shear reinforcement			
		Anchorage Type	ρ_s (%)	S_h (mm)	ρ_h (%)	S_v (mm)	ρ_v (%)
A5FF	0.5	90° hook	0.89	110	0.8	110	0.8
A10FF	1.0						
M5FF	0.5	Mechanical anchorage					
M10FF	1.0						
M5NN	0.5						
M10NN	1.0						
M5FN	0.5					0.0	0.0
M10FN	1.0						
M5NF	0.5			0.0	0.0	110	0.8
M10NF	1.0						

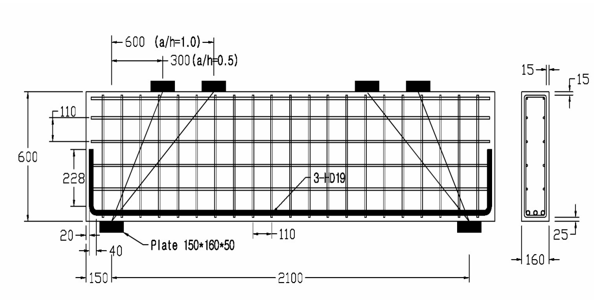

(a) standard 90-degree hook


(b) Mechanical anchorage

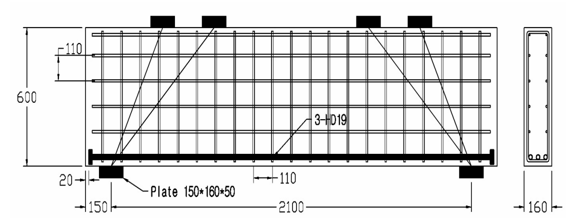
* First letter states anchor type of reinforcement (A; ACI standard hook, M; mechanical anchorage). Number is shear span-to-height ratio a/h. Letters after number means web reinforcement detail type

Test setup and instrumentation

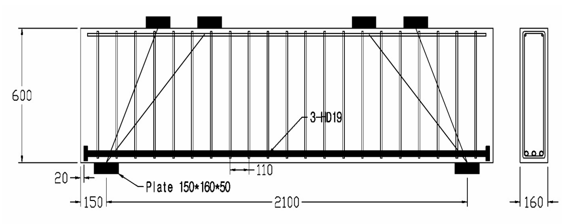
All beams were tested to failure under two-point symmetric top loading as shown in Fig. 3. Vertical deflections were monitored by the LVDTs. At each load increment, the test data were captured by a data logger and automatically stored. The strains of reinforcements were measured using 5mm strain gages. Until the first crack occurred, load was applied by keeping increments of 20kN. Subsequently, the load increments were increased to 40kN each after the crack. Applied loads and support reactions were transmitted to the specimens by means of 150×160×30mm steel plates.



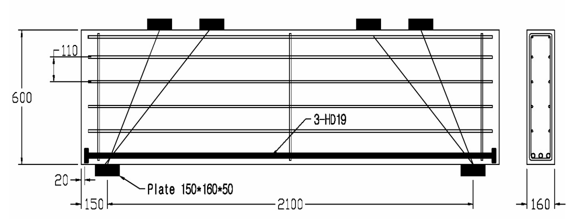
A5FF, A10FF



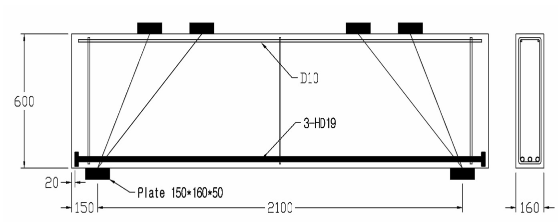
M5FF, M10FF



M5NF, M10NF



M5FN, M10FN



M5NN, M10NN

Fig. 1 Reinforcement details of experimental deep beams

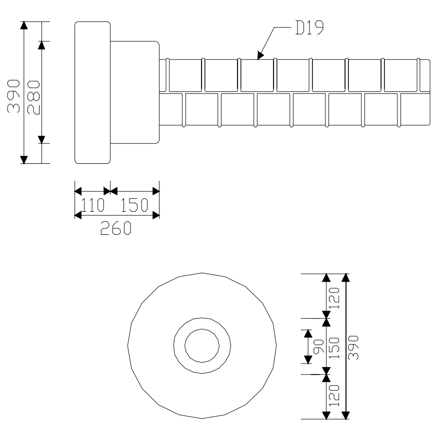


Fig. 2 Details of mechanical anchorage

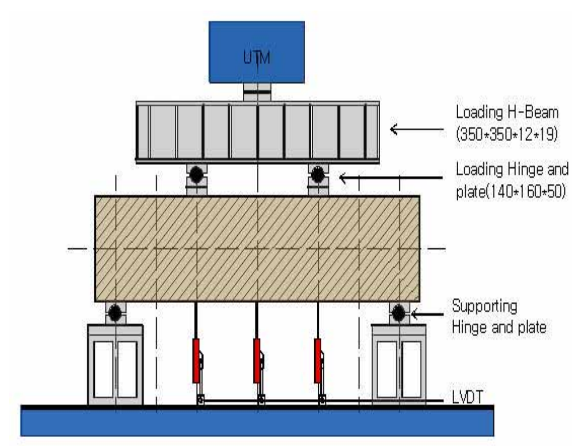


Fig. 3 Specimen test setup

Table 2 Failure modes and experimental results

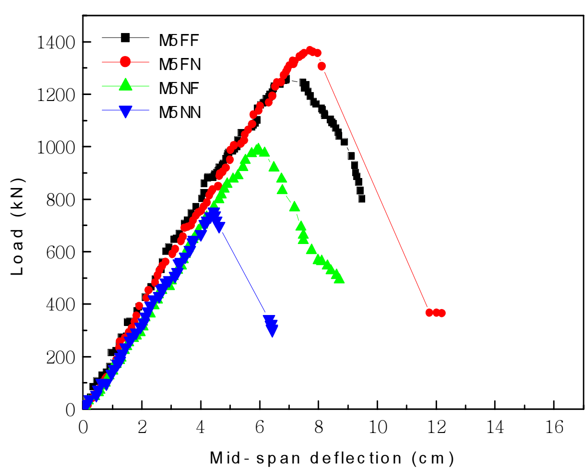
Name	Measured shear strength (kN)		Measured Shear stress (N/mm^2)		δ_u (mm)	δ_{cr} (mm)	Measured strain at Vu (×10-6)			Slope of diagonal crack (°)	Failure Mode
	V_u,	V_{cr}	v_u	v_{cr}			ε_s	ε_h	ε_v		
A5FF	591	171	6.16	1.78	50	21	866	4800	292	64	Bearing failure
A10FF	458	161	4.77	1.68	97	33	2929	9812	9521	43	Diagonal-splitting
M5FF	629	230	6.55	2.40	69	22	1282	7259	1258	60	Bearing failure
M10FF	508	161	5.29	1.68	131	44	3027	2511	3276	48	Shear-compression
M5NN	376	132	3.92	1.38	44	17	1072	-	-	65	Bearing failure
M10NN	329	141	3.43	1.47	65	31	1750	-	-	50	Diagonal-splitting
M5FN	683	294	7.11	3.06	59	24	1589	8977	-	70	Shear-compression
M10FN	371	156	3.86	1.63	73	28	1794	2928	-	46	Shear-compression
M5NF	494	220	5.15	2.29	73	26	1038	-	1220	60	Diagonal-splitting
M10NF	422	166	4.40	1.73	78	30	2262	-	8586	43	Diagonal-splitting

Where V_u is shear strength at peak point and V_{cr} is shear strength at initial diagonal crack; v_u is shear stresses at peak point and v_{cr} is shear stress at initial diagonal crack; δ_u, δ_{cr} are deflections at V_u and V_{cr}, respectively; ε_s, ε_h, ε_v are strains of main flexural reinforcement, horizontal shear reinforcement and vertical shear reinforcement, respectively.

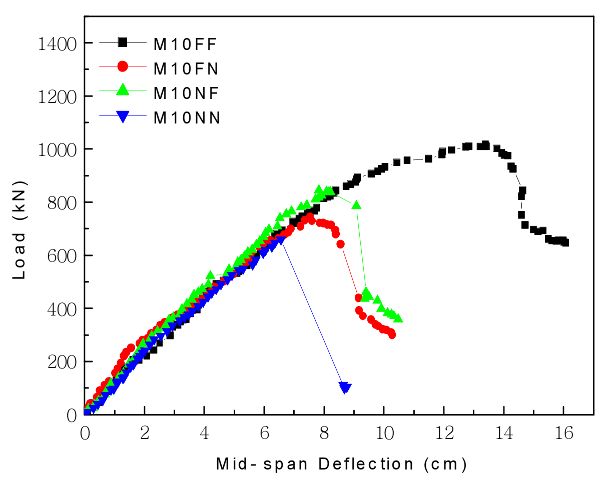
BEHAVIOR OF TEST SPECIMENS

Load-displacement relationship

Fig. 4 shows the mid-span deflections of specimens with different shear span-to-overall height ratio *a/h*. All specimens with the same *a/h* had a similar initial stiffness but different after diagonal crack. The initial diagonal cracks were found at 32%-42% of ultimate shear strength in the specimen with *a/h* of 0.5, while at 34%~48% in specimen with *a/h* of 1.0. The mid-span deflections decreased with an increasing amount of web reinforcement. On comparing ductility among all specimens, specimens M5FF and M10FF which had both vertical and horizontal shear reinforcement for crack-control according to ACI 318-02 Appendix A showed more ductile behavior after yield than others. After diagonal crack, the shear stiffness of the beams without web reinforcement dropped significantly.



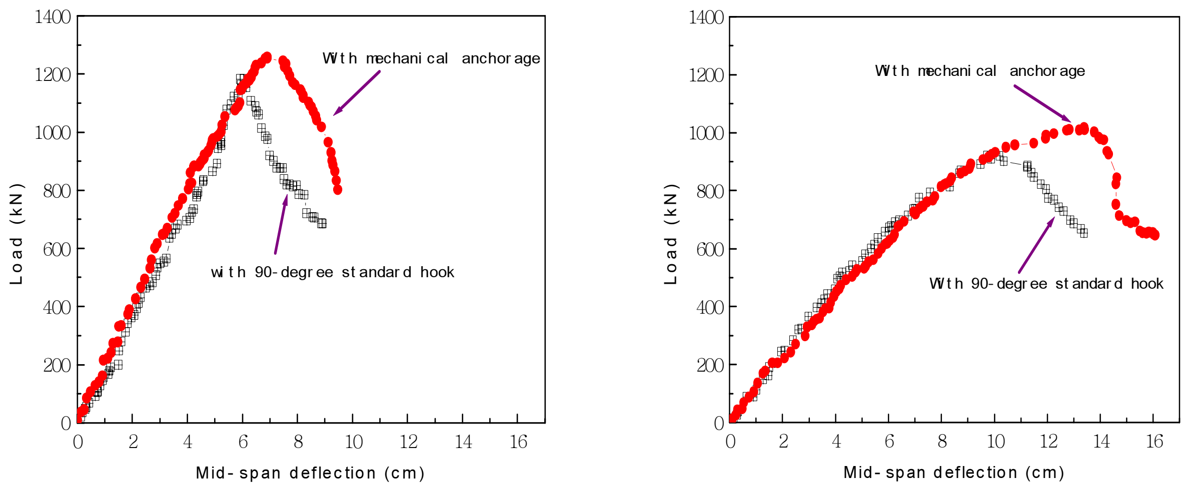
(a) Specimen with *a/h* of 0.5



(b) Specimens with *a/h* of 1.0

Fig. 4 Load-deflection curve of specimens with mechanical anchorage

Load increment patterns of each specimen were similar until ultimate load without the influence of anchorage type. In case of *a/h* of 1.0, however, specimens with mechanical anchorage showed more ductile behavior than specimen with ACI 90-degree standard hook after ultimate strength. At ultimate strength, specimen H5FF with mechanical anchorage showed 6.5% higher strength than specimen A5FF with 90-degree standard hook. In *a/h* of 1.0, similarly, the strength of specimen M10FF with mechanical anchorage was 10% higher than that of A10FF. On the viewpoint of initial shear crack formation, there were not severe differences between specimens with mechanical and hook anchorage in *a/h* of 1.0. In case of *a/d* of 0.5, however, the formation of initial shear crack was delayed by using the mechanical anchorage. This result is supposed to be originated from the strong anchorage capacity of mechanical anchor. From this, it can be concluded that the mechanical anchorage has high anchorage capacity and can sufficiently be applied to deep beam when it satisfy the development length requirement of the ACI.



(a) Specimen with a/h of 0.5

(b) Specimens with a/h of 1.0

Fig. 5 Comparison of load-deflection curve for specimens with different anchorage type

Crack patterns and Failure modes

Fig. 6 shows the crack patterns at failure for all 10 deep beams together as well as the loads at which each crack was first observed. Failure of all specimens took place only after the primary diagonal crack developed fully between the load and support region, and after yielding of main tension reinforcement. Primary diagonal crack parallel to the axis of the compression struts was observed for all specimens. It was found that the specimen with both vertical and horizontal reinforcements had more scattered crack pattern than the specimen with either horizontal or vertical reinforcement. The specimen with mechanical anchorage showed con failure of concrete that is one of typical failure pattern of anchor around vicinity of anchorage.

The final failure mode in this experiment could be classified into three types of failure pattern; diagonal splitting failure, compression strut failure and shear compression failure. In the specimen M10NN, as a the diagonal splitting failure pattern, shear cracks connected with loading and supporting point occurred and expanded to failure making a booming sound. The compression strut failure was found in the specimen M5FN in which compression struts were formed due to several diagonal cracks and finally developed to the failure of at upper or lower part of strut. The specimen M5FF showed the local compression failure causing collapse of the specimen at around the loading point.

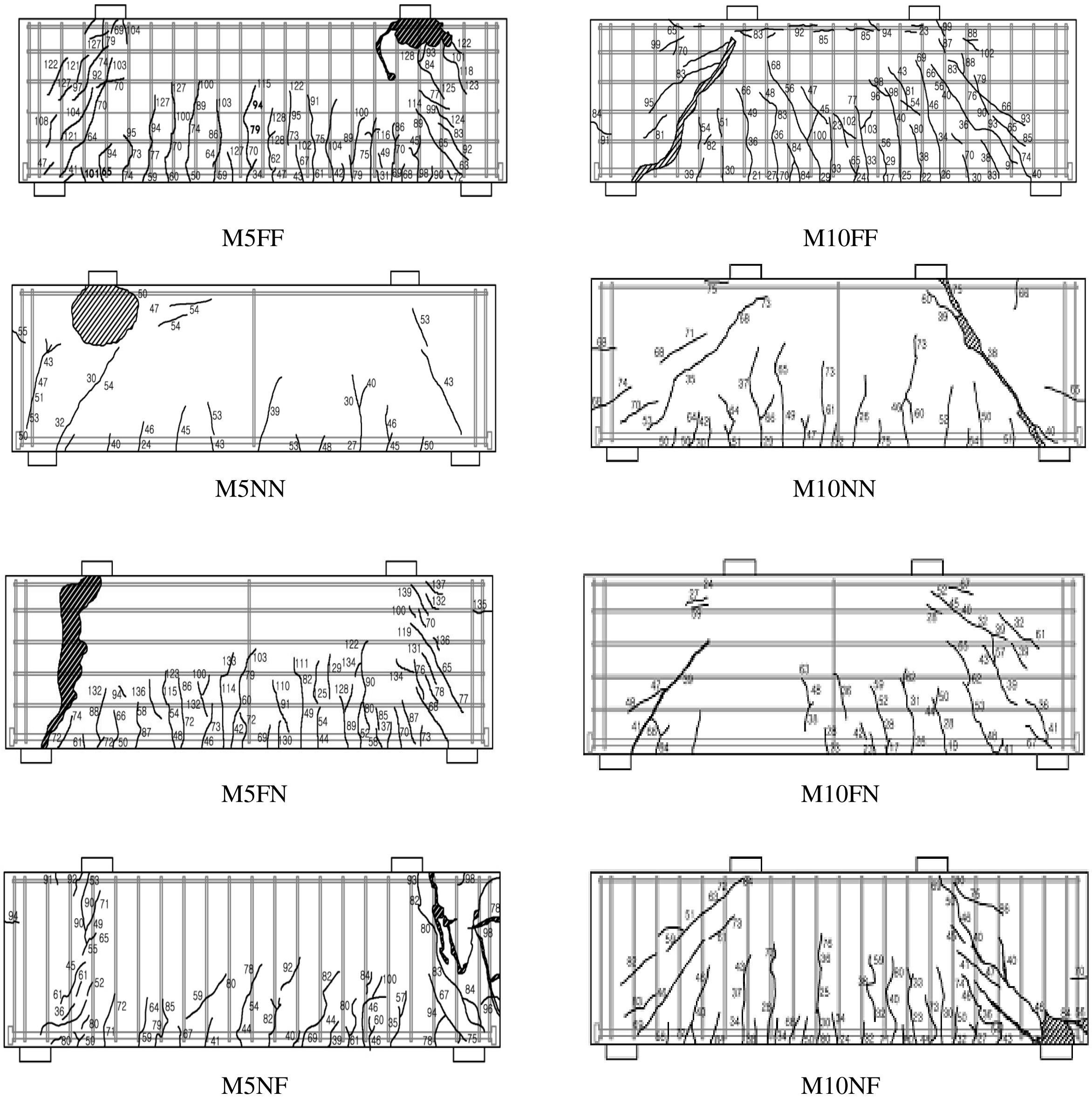
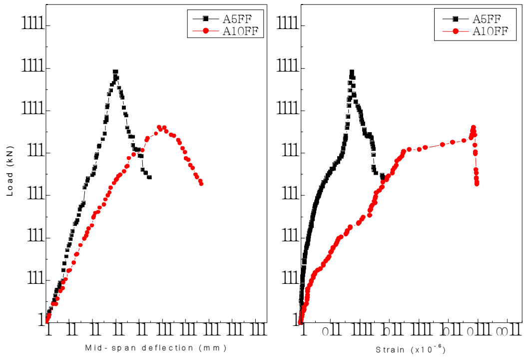


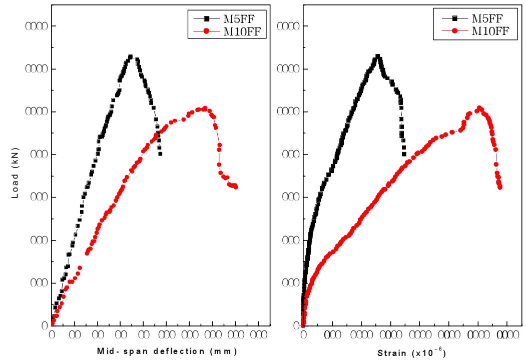
Fig. 6 Crack patterns at failure

Effect of shear span-to-overall height ratio

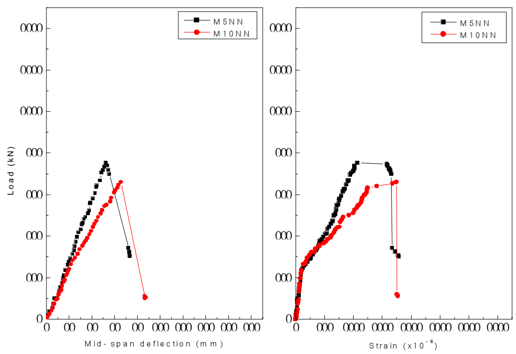
Fig. 7 shows the load-deflection curves and the load-strain curves of longitudinal reinforcement for specimens with different *a/h*. The ultimate shear strength of tested specimen decreased when *a/h* increased as shown Fig. 7. From the comparison of A-series (A5FF, A10FF) and M-series (M5FF, M10FF) with both horizontal and vertical shear reinforcements, it was found that the deflection and strain increased from 65 to 94% and from 136 to 240% with increment of *a/h*, respectively. Specimens M5NN and M10NN without web reinforcement showed increased deflection and strain of 25% and 63%, respectively at *a/h* of 1.0. In specimen M5FN and M10FN with horizontal shear reinforcement only, however, the deflection was decreased about 2.4% and the strain of longitudinal reinforcement increased about 13% when *a/h* increased. The comparison of specimen M5NF and M10NF specimen with vertical shear reinforcement only showed that the deflection and the strain increased about 5% and 118% with the increment of *a/h*, respectively In addition, with *a/h* increasing, the tied-arch action becomes less effective because of the reduced angle.



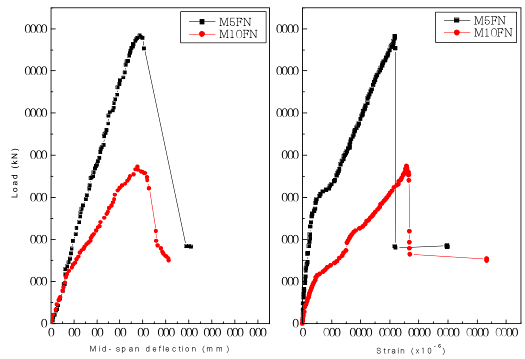
(a) Specimens with 90-degree hooks



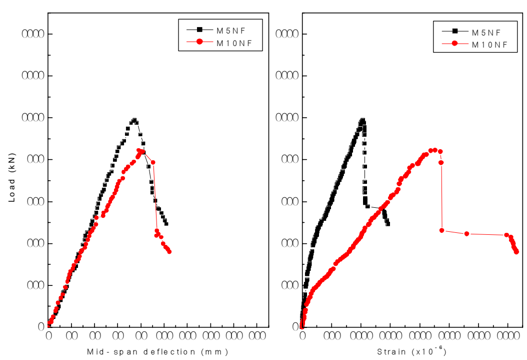
(b) Specimens with mechanical anchorages



(c) Specimens without web reinforcements



(d) Specimens with horizontal reinforcements only



(e) Specimens with vertical reinforcements only

Fig. 7 Load-deflection and strain curves with variable *a/h*

Strain in web reinforcements

Strains measured in the horizontal and vertical shear reinforcements at the critical section were shown in from Fig. 8 to Fig. 11. From Fig. 8 and Fig. 10, it can be seen that there are not clear differences of strains between horizontal and vertical shear reinforcements before the diagonal crack. However, after the crack, the strain of horizontal reinforcement rapidly increased while vertical reinforcement showed a little extension of strain. This means that the contribution of horizontal reinforcement is to be high in specimens with *a/h* of 0.5 after crack. In specimens A10FF, M10FF with *a/h* of 1, the serious difference of strain between the horizontal and vertical reinforcement was not found until failure. From this, it can be concluded that both two reinforcements develop similar contribution to the shear at *a/h* of 1.

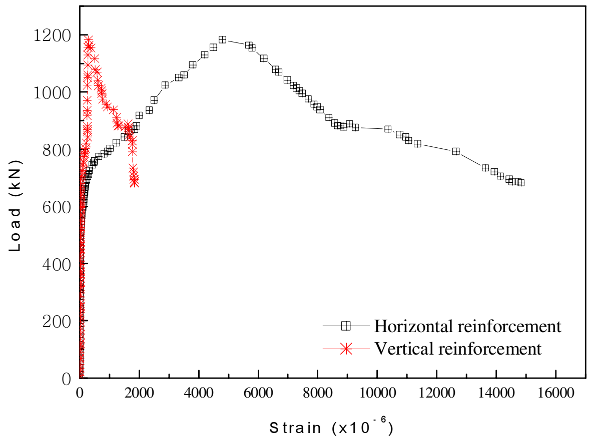


Fig. 8 Load-strain curve of specimen A5FF

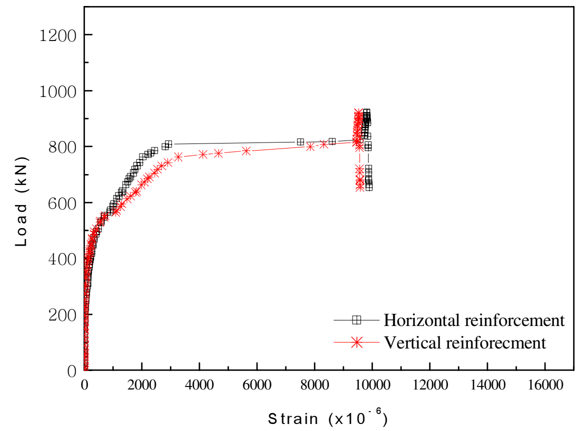


Fig. 9 Load-strain curve of specimen A10FF

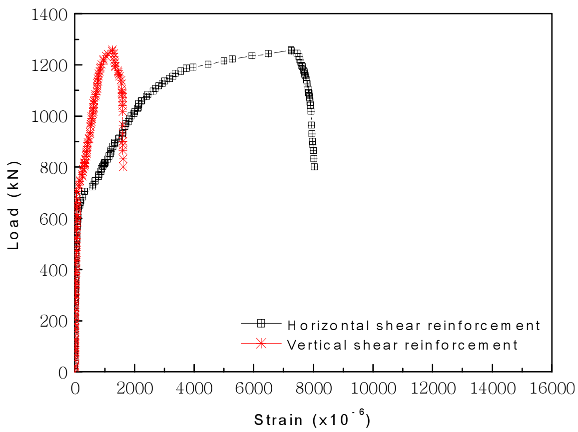


Fig. 10 Load-strain curve of specimen M5FF

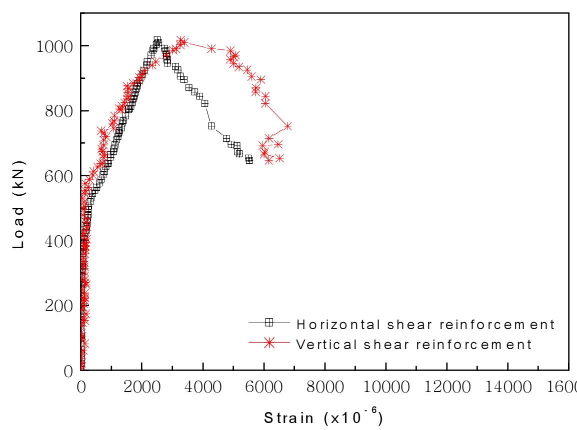


Fig. 11 Load-strain curve of specimen M10FF

EVALUATION ON CODE DESIGN FORMULAS FOR SHEAR

ACI 318-99 Section 11.8[4)]

In ACI 318-99 Code, the sectional shear strength of deep flexural member is calculated by combining the contributions of both concrete and distributed shear reinforcements. Ultimate shear strength by concrete and reinforcement are shown in from Eq. (1) to Eq. (3). The concrete contribution can be counted by Eq. (1) or Eq. (2).

$$V_c = 2\sqrt{f_c{}'}b_w d \quad \text{(psi, in.)} \quad \text{(Eq. 11-28)} \qquad (1)$$

$$V_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left(1.9\sqrt{f_c{}'} + 2500\rho_w \frac{V_u d}{M_u}\right)b_w d \le 6\sqrt{f_c{}'}b_w d \quad \text{(psi, in.)} \quad \text{(Eq. 11-29)} \qquad (2)$$

where $3.5\text{-}2.59(M_u/V_u d)$ is to be kept less than or equal to 2.5; and f'_c = specified compressive strength of concrete; b_w = web width; A_s= area of nonprestressed tension reinforcement; d =distance from extreme compression fiber to centroid of longitudinal tension reinforcement; V_u=factored shear force at critical section; ρ= ratio of tension reinforcement; l_n= clear span; a = shear span; M_u = factored moment occurring simultaneously with V_u at the critical section.

The use of shear reinforcement is required whenever the factored shear force at the critical section exceeds the shear strength by concrete. The contribution from the shear reinforcement is computed with Eq. (3).

$$V_s = \left[\frac{A_v}{s} \left(\frac{1 + \frac{l_n}{d}}{12} \right) + \frac{A_{vh}}{s} \left(\frac{11 - \frac{l_n}{d}}{12} \right) \right] f_y d \quad \text{(psi, in.)} \quad \text{(Eq. 11-30)} \quad (3)$$

where A_v = area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s; A_{vh} = area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2.

The ACI 318-99 Code defines an upper limit for the shear strength of deep flexural members as shown in Eq. (4).

$$V_n = \begin{cases} 8\sqrt{f_c'}b_w d & for \quad l_n / d < 2 \\ \frac{2}{3}\left(10 + \frac{l_n}{d}\right)\sqrt{f_c'}b_w d & for \quad 2 \le l_n / d \le 5 \end{cases} \quad \text{(Eq. 11-27)} \quad (4)$$

By virtue of study of Gerardo Aguilar[1)] and Kang-Hai Tan[6,7)], it has been found that the role of horizontal steel contribution is overestimated in the formulas for deep beam design in ACI 318-99 Code.

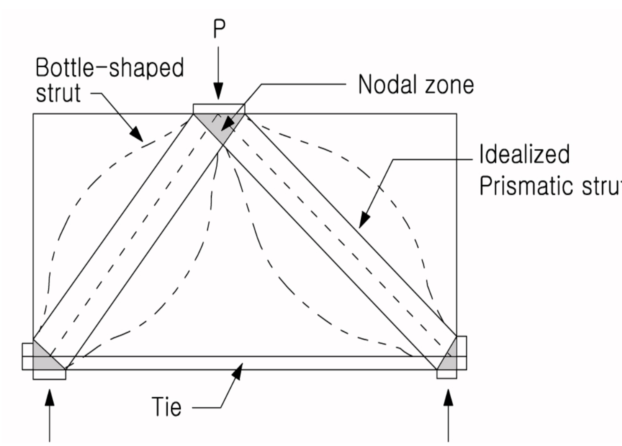


Fig. 12 Description of Strut-and-Tie Model

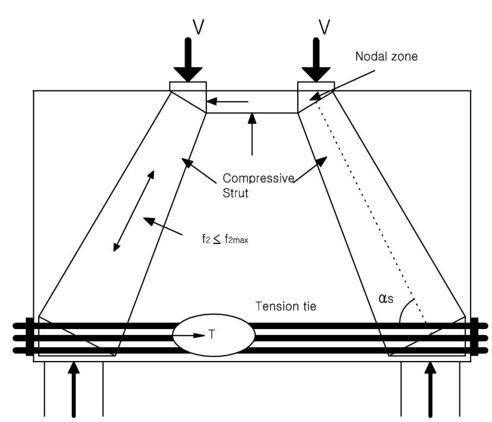


Fig. 13 Description of Canadian Code

Appendix A of ACI 318-02 Building Code[3)]

Appendix A of ACI 318-02 code provides new approaches to the shear design of deep beam. In the Strut-and-Tie Model (STM) approach, the flow of forces or stresses within the member is represented by means of a truss like Fig. 12. STM consist of the struts, ties and nodal zones. The permitted stress of all struts, ties and nodal zones shall not exceed the limited value.

The effective compressive strength of the concrete in strut and nodal zone shall be taken as Eq. (5) and Eq. (6).

$$\text{Strut}: f_{cu} = 0.85\beta_s f_c' \text{ (psi) (Eq.(A-3))} \quad (5)$$

$$\text{Nodal zone}: f_{cu} = 0.85\beta_n f_c' \text{ (psi) (Eq.(A-8))} \quad (6)$$

where the value β_s and β_n range 0.4～1.0 and 0.6～1.0, respectively.

In case that concrete compressive strength is less than 41MPa, the minimum steel quantity for preventing crack can be increased 25%, if it is satisfied with a minimum amount of the grid reinforcement crossing the strut, or required steel in same direction as shown in Eq. (7).

$$\sum \frac{A_{si}}{bs_i} \sin \gamma_i \geq 0.003 \quad \text{(Eq.(A-4))} \quad (7)$$

where A_{si} is area of surface reinforcement in the i^{th} layer crossing a strut and b is width of beam and S_i is spacing of reinforcement in the i^{th} layer adjacent to the surface of the member.

Canadian CSA A23.3-94[5)]

In Canadian CSA Code, STM is recommended as a method for shear design of deep beam. Unlike the ACI Code, the 1984 CSA Code uses the concept of shear span-to-effective depth ratio *a/d* rather than the effective span-to-depth ratio l_n/d for deep beam design. The CSA Code permits two alternative design methods for shear, the simplified method and the general method. The latter is based primarily on the Modified Compression Field (MCF) theory developed by Collins et. Al[8)]. The compressive stress in the inclined concrete strut shall be based on Eq. (8).

$$f_{cu} = \frac{f_c'}{0.8 + 170\varepsilon_1} \leq 0.85 f_c' \quad \text{(MPa) (CSA 11.30)} \quad (8)$$

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s \quad \text{(CSA11.31)} \quad (9)$$

where ε_1 = the principal tensile strain in cracked concrete due to factored loads; α_s = the angle between the strut and the adjoining tensile ties; ε_s = the tensile strain in the tensile tie inclined at as to the compressive strut.

Comparison of test results with design formulas

A comparison between the measured and calculated shear strength for the tested specimens was carried out. The codes considered for the comparison are the ACI 318-99, the Appendix A STM of ACI 318-02 and the CSA A23.3-94. The material safety factors for concrete and steel reinforcement have been set to unity for comparison purpose. Table 3 and Fig. 14 show the comparison. The shear design procedures of the ACI 318-99 and STM of the ACI 318-02 underestimated the shear strength of deep beam, especially showed lowest predicted shear strength for the specimen with horizontal shear reinforcement only. In the shear strength prediction of deep beams, the ACI 318-99 has the lowest average mean of 0.65 among the three methods. Using the shear design method of CSA, it was shown that most close prediction could be achieved. In case of the specimen without web reinforcement, however, CSA Code overestimated shear strength by including the effect of web reinforcements that do not exist and considering maximized effective stress of strut. The ratio between results of STM and test, V_{STM}/V_{TEST} is ranged at an average mean of 0.74 and the standard deviation of 0.10. This means that STM is recommended as a most desirable method because it has lowest standard deviation although its predicted shear strength is relatively higher than that of ACI 318-99.

Table 3 Summary of predictions for ultimate shear strength of deep beams

Specimen	V_{TEST} (kN)	V_{ACI} (kN)	V_{STM} (kN)	V_{CSA} (kN)	$\frac{V_{ACI}}{V_{TEST}}$	$\frac{V_{STM}}{V_{TEST}}$	$\frac{V_{CSA}}{V_{TEST}}$
A5FF	591	348	431	569	0.59	0.73	0.96
A10FF	458	348	309	399	0.76	0.67	0.87
H5FF	629	348	431	569	0.55	0.69	0.90
H10FF	508	348	309	399	0.69	0.61	0.79
H5NN	376	221	345	521	0.59	0.92	1.39
H10NN	329	193	248	472	0.59	0.75	1.43
H5FN	683	348	431	569	0.51	0.63	0.83
H10FN	371	328	309	399	0.88	0.83	1.08
H5NF	494	309	431	569	0.63	0.87	1.15
H10NF	422	283	309	399	0.67	0.73	0.95
				MEAN	0.65	0.74	1.04
				STDEV	0.11	0.10	0.23

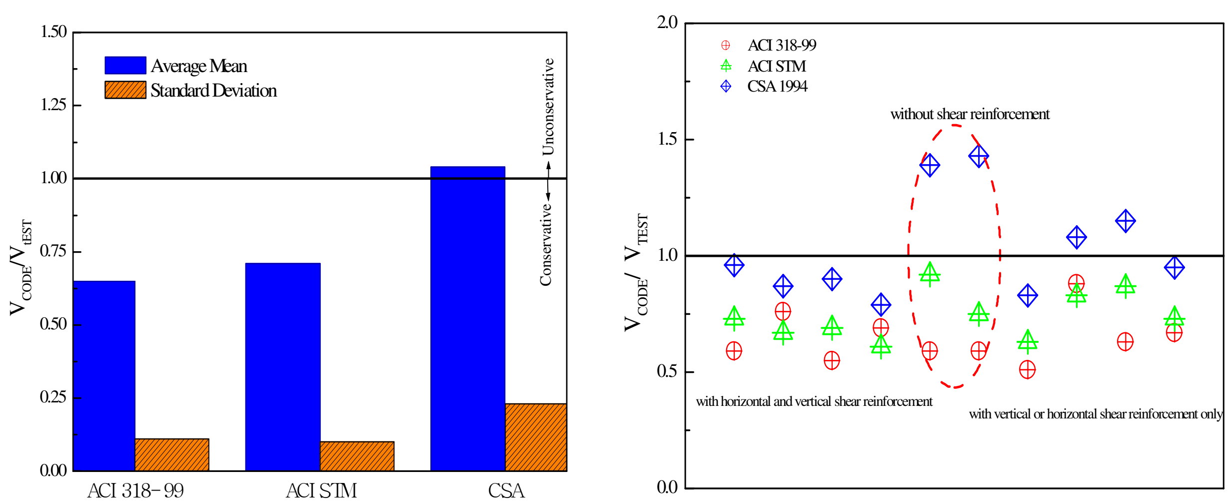


Fig. 14 Comparison of shear strength prediction by different methods

CONCLUSION

1. On comparing the case with 90-degree hook anchorage, the specimen with headed reinforcements as a mechanical anchorage showed better load resistance capacity when it was designed to satisfy the development length requirement of the ACI code.
2. The deep beam with the shear reinforcements satisfying the requirement of equation (A-4) of ACI 318-02 Code showed effective behavior for crack control and ductile behavior after yield. In case of the specimen with only one directional shear reinforcement or nothing, however, it was shown that

shear strength rapidly decreased after the ultimate shear strength.

3. For most specimens, the load was supported by compression strut connecting with loading point and bearing point at failure. And it was destroyed after the formation of diagonal crack paralleled with struts showing brittle fracture when the diagonal splitting or compression of strut governed the failure of specimen.
4. In case of shear span-to-overall height ratio *a/h*=1, the strains of vertical and horizontal shear reinforcement were similar after yield of longitudinal reinforcement. But in case of *a/h*=0.5, horizontal shear reinforcement showed higher strain than vertical shear reinforcement after formation of initial diagonal cracks. From this, it can be found that the horizontal reinforcement has higher contribution than vertical reinforcement to the shear strength when the *a/h* is low.
5. In predicting shear strength of deep beam, Strut-and-Tie Model of Appendix A in ACI 318-02 was conservative and showed lowest standard deviation among several design methods**.** Therefore, it was judged that STM is a most desirable method for the design of deep beam.

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