- Technical Paper -

EVALUATION ON SHEAR FAILURE MECHANISMS OF A RC COLUMN WITH CUT-OFF BARS BASED ON E-DEFENSE EXCITATION

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ABSTRACT

Experimental tests on a RC column with cut-off longitudinal bars (LG) was performed. In this study, the damage condition and the failure mechanisms were summarized and evaluated in detail. It was found that due to flexural yield of several LG bars at upper cut-off position, the horizontal cracks developed through until about half of the section firstly at the just point of upper cut-off position. The cracks then extended downwards into diagonal shear cracks. Thus, with greater angle (60°) to vertical axis compared to standard 45° assumption in specification, shear resistance was reduced greatly, because of smaller concrete area and less hoops to resist shear. As a result, severe shear failure occurred to the range below the just point of the upper cut-off position.

Keywords: E-Defense, RC column, cut-off LG bars, failure mechanisms

1. INTRODUCTION

To reduce amount of longitudinal reinforcement at the section where it is unnecessary by distributed moment, it was common practice until the mid-1980s to cut off LG bars in RC columns. However, this type of piers was extensively damaged during 1978 Miyagi-ken -oki Earthquake, 1982 Urakawa-oki Earthquake and 1995 Kobe Earthquake. Its failure mechanisms were widely studied. Study[1] proposed an inspection method to identify its vulnerability, focusing on effect by development length. It was understood that this type of column failed without sufficient development length. The shear resistance of this type of columns is normally determined by concrete and confinement (hoops for pier) separately[2]. The resistance by concrete is obtained by assuming the shear develops along 45° to the vertical axis. For RC columns with cut-off LG bars, however, lateral cracks due to flexure may occur first, and leads to further shear cracks. This phenomenon may further cause the degrading of shear resistance by concrete and earlier failure of the columns with cut-off of LG bars. Besides, study[3] based on experimental tests showed that the scale effect had notable influence on its behavior.

Detailed progress of damage, relation between flexural and shear failure, and mechanisms of shear resistance, especially at the cut-off position are still not understood in detail. The full-scale experimental test on RC column with cut-off LG bars (specimen No. C1-2 shown in Fig.1) by E-Defense excitation provides researchers the opportunities to understand the failure mechanisms close to the actual failure. Thus, in this study, the failure mechanisms will be made clear. Based on the modified compression field theory, the developed angle of cracks is considered and the degrading of shear resistance is also evaluated in detail.

2. EXPERIMENTAL SETUP OF C1-2 SPECIMEN

2.1 Experimental Setup

As shown in Fig. 2, Specimen C1-2, a typical reinforced concrete column built in the 1970s (7.5 m high, 1.8 m diameter), was assumed to be damaged by shear. It was designed as a full-scale model based on a combination of static lateral force method and working stress design based on 1964 Design Specification of Steel Road Bridges[4], with 2 layers of cut-off LG bars. As shown in Fig. 2, the column has totally 3 layers of LG reinforcing bars at base with 29mm diameter, respectively 32, 32 and 16 bars at outer, middle and inner layers. The middle layer was cut off at height of 3.86m (namely upper cut-off point) and the inner at height of 1.88m (namely lower cut-off point). Deformed circular

Fig. 1 Experimental Test on C1-2 Specimen

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hoops with diameter of 13mm were arranged at 300mm intervals, except outer hoops at top 1.15m zone and at base 0.95m zone at 150mm intervals. Stirrups are lap spliced with 390mm (30 times its diameter). As a consequence, the LG bar ratio is 2.02% beneath the lower cut-off point, 1.62% beneath the upper cut-off point, and 0.81% above the upper cut-off point, while the volumetric ratio of hoop is 0.106% for middle and 0.422% for and base. Based on material tests, the average yield strength of LG bars and hoop bars are 383 MPa and 409 MPa, and the elastic modulus of them are 207 GPa and 196 GPa respectively. Therefore, their yield strain was 1850μ and 2050μ. Additionally, the compressive strength of concrete is 30.8 MPa based on material test.

As shown in Fig. 1 for the general setup of experiment, two simply supported steel decks were set upon the RC column and two steel bents (side bents). Four mass blocks (245.2t in total) were fixed on decks to simulate the dead load. Between deck and column, 2 fixed bearings (fixed in both longitudinal and transverse directions) and 4 sliders (only providing upward support) were set. The footing of C1-2 specimen was anchored as fix on shake table of E-Defense. The table was excited using E-Takatori ground motion (modified from the observed Takatori ground motion by taking the soil-structure interaction based on FEM analysis into consideration) in 3 directions. Main excitation using 100% E-Takatori ground motion was conducted only once, since great failure have occurred to the column.

2.2 Design Resistance and Assumed Damage

To assess the failure pattern, the development length l_d, and the shear resistance V_R is defined as follows according to JSCE[5]:

$$l_d = \frac{\left(\dfrac{f_y}{1.25\sqrt{f_c}} - 13.3\right)\phi}{0.318 + 0.795\left(\dfrac{c}{\phi} + \dfrac{1.5A_t}{s\phi}\right)} \tag{1}$$

$$V_R = V_C + V_S \tag{2}$$

where f_y: yield strength of LG bars; f_c: strength of concrete; c: minimum of thickness of cover concrete and half of distance between LG bars; s: distance between hoops; ϕ: diameter of LG bars; A_t: area of reinforcement perpendicular to assumed failure surface; V_C: shear resistance by concrete; V_S: shear resistance by steel reinforcement.

The development length for upper cut-off is 440mm (15.2Φ). Correspondingly, the zone just above the point of cut-off, the zone in development length, and the zone below development length, is defined as Section I, II and III respectively (Fig. 2). Besides, the proposed shear strength by Kawano[6] is applied to concrete. With the flexural resistance, the resistance distribution along height can be drawn in Fig. 2 (b), and the P-δ relation can be drawn in Fig. 3[7]. Accordingly, the assumed failure events are listed as: (1) Flexural yield of LG bars in Section II; (2) Shear failure in Section I; (3) Flexural ultimate stage of Section II.

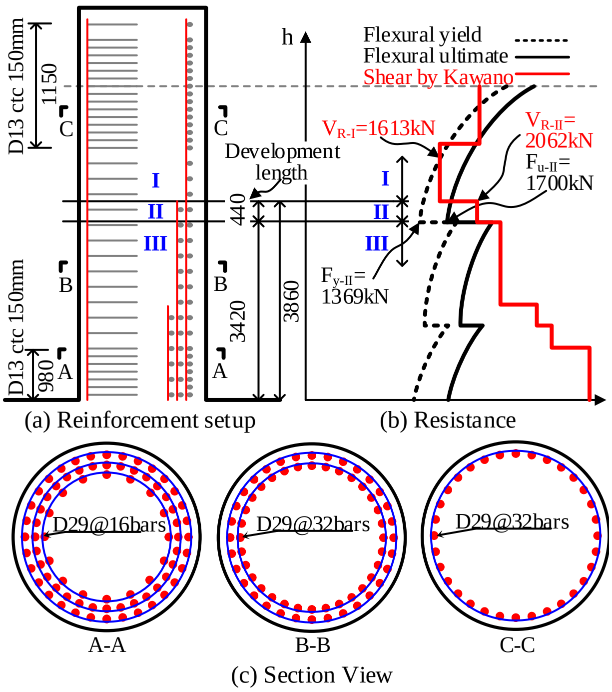


Fig. 2 Experimental Setup and Resistance Distribution

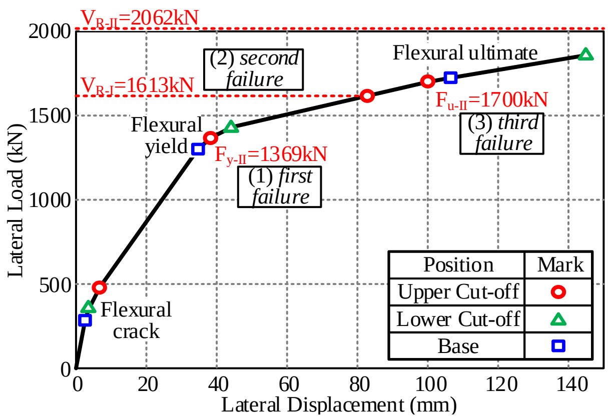


Fig. 3 Design Resistance

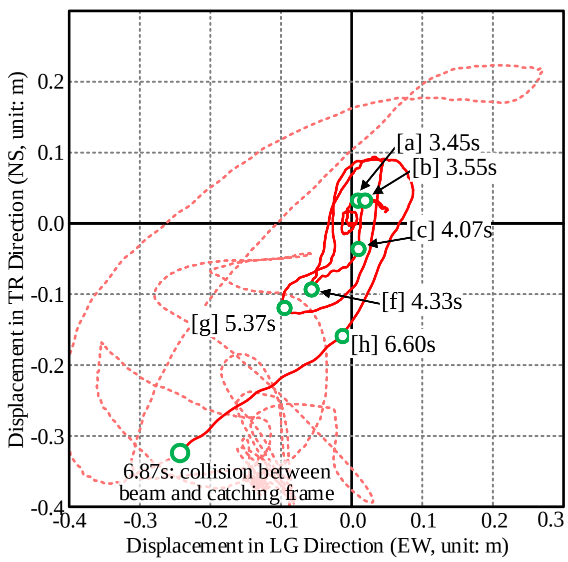


Fig. 4 Response Displacement History on Column Top

3. EXPERIMENTAL RESPONSE

3.1 General Experimental Response

Subjecting to E-Takatori excitation, the response displacement orbit at the top of column (height of 7.5m) is shown in Fig. 4. It should be noted that NS and EW directions correspond to the transverse (hereafter as TR) and LG directions respectively of the model. The main response of the column was in the SW-NE direction.

Key time points of important damage events are shown in Fig. 4 and listed in Table 1 (also refer to Fig. 5 and Fig. 6). The first yield of LG bars occurred at about [a] 3.45s at base of column, followed by the first yield of LG bars at upper cut-off point at [b] 3.55s. The orbit figure shows that the displacement at these time points was still not great. Then, a visible horizontal crack was firstly developed from [c] 4.07s to [d] 4.12s along NW to E surfaces at height of about 3.9m (position near the upper cut-off point). After reaching the temporary peak resisting load (1418 kN) at [e] 4.26s (refer to Fig. 6), these horizontal cracks developed into diagonal cracks until [f] 4.33s (also the temporary peak displacement).

Then, the column kept vibrating, resulting in following horizontal cracks possibly due to flexural, and diagonal cracks possibly due to shear. After reaching the following peak displacement at [g] 5.37s, the spalling-off of the covering concrete began to occur at about 6.04s at N and NW surfaces near the upper cut-off point, and this developed as well at S and SW surfaces at about [h] 6.60s. Afterwards, the column continuously responded toward SW direction with spalling off of covering concrete and the bottom of lateral beam at the edge collided with the catch system at about [i] 6.87s. As this may influence the response of the column, the following discussion is mainly focused on the time point up to this collision.

3.2 Failure of Column based on Experimental Test

To evaluate the failure, the history of combined displacement is plotted in Fig. 5 for the time period from 2.0 sec to 6.87 sec. It becomes obvious to us that the first shear crack occurred at one peak displacement at 4.33 sec, after the flexural crack occurred around 4.07 sec. Following peak greater than that at 4.33 sec occurred at 5.37 sec. Then, great failure occurred at upper cut-off point and column base and collision between the lateral beam and catch system occurred.

Similarly, the load-displacement (P-δ) history is illustrated in Fig. 6. Here, both the displacement and the lateral load are calculated into the combined direction based on measured results in separated LG and TR direction. Especially, the history from 4.00 ~ 4.50s is highlighted. It can be observed that the column behaved roughly according to the design P-δ (based on flexure), until the occurrence of the first horizontal crack at [c] 4.07s, although the stiffness is slightly smaller after the first yields of LG bars at the base of column (at [a] 3.45s) and the upper cut-off point (at [b] 3.55s). In spite of the sudden decrease just after [c] 4.07s, the lateral load increased again and got to about same value until [d] 4.12s, at when the horizontal cracks began to develop in

Table 1 General Damage Events

Time Point	Response and Failure		
	Base	Upper Cut-off	General
[a] 3.45s	1st yield of LG bar		
[b] 3.55s		1st yield of LG bar	
[c] 4.07s		1st visible horizontal crack + yield of half LG bar	
[d] 4.12s			
[e] 4.26s		1st diagonal crack + 1st yield of ties	Peak load
[f] 4.33s			Load drop + Peak disp
[g] 5.37s		Opposite diagonal crack	Peak disp
[h] 6.60s		Great failure	
[i] 6.87s		Collision between lateral beam and catching frame	

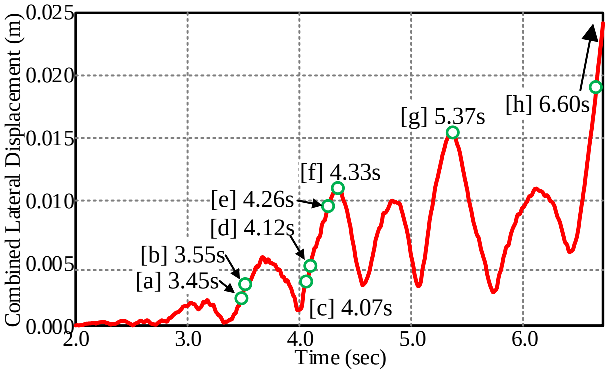


Fig. 5 Response History of Combined Lateral Displacement at Top of Column

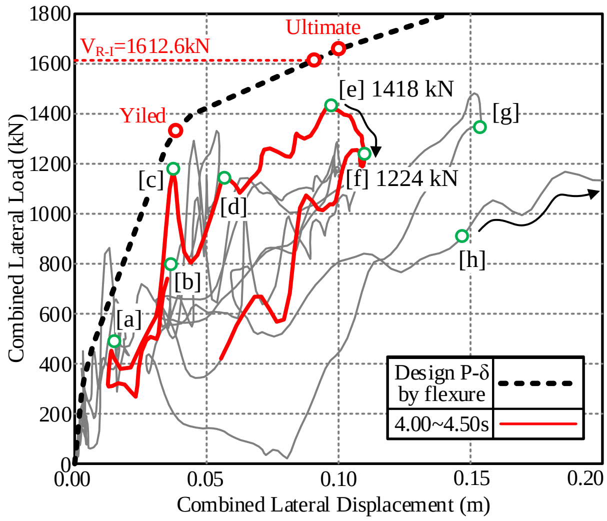


Fig. 6 Response Load-Displacement History on Top of Column

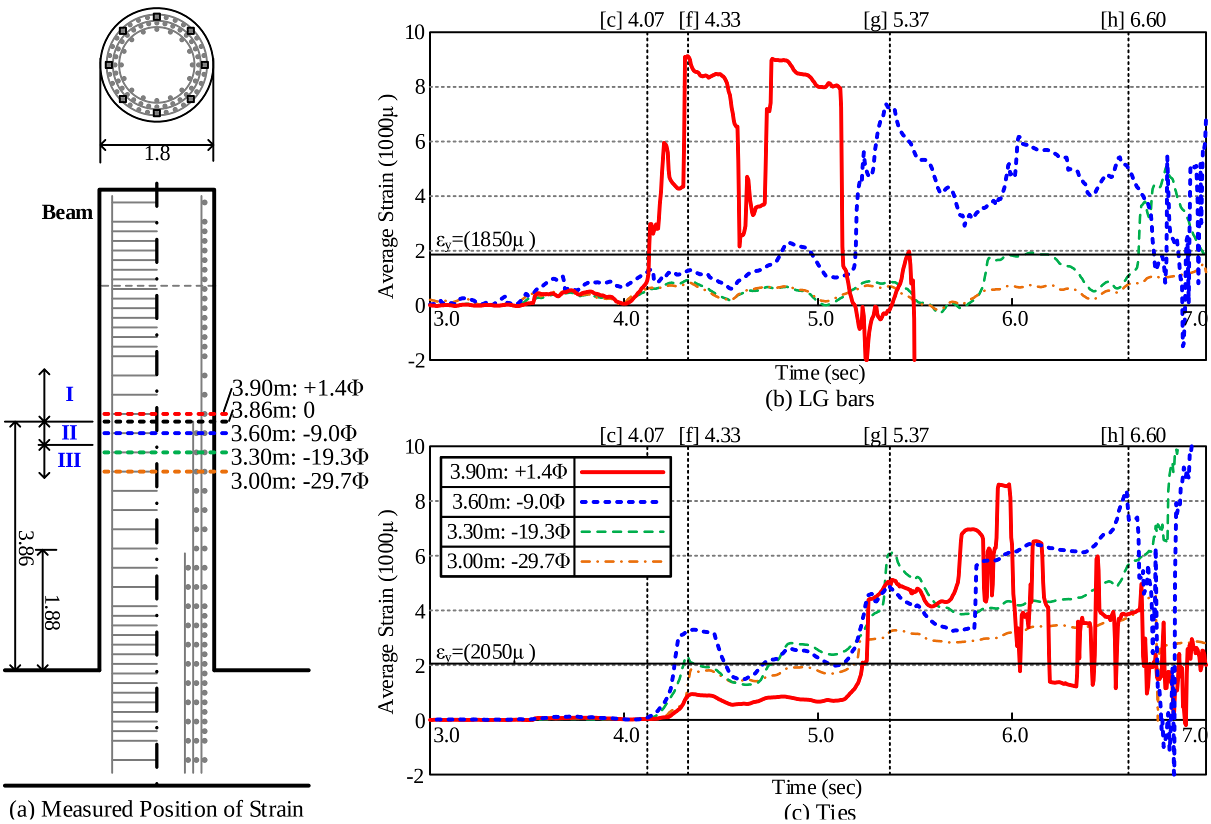


Fig. 7 Response Histories of Strain in LG Bars and Hoops around Upper Cut-Off Position

diagonal direction. Then, the lateral load increased gradually to the temporary peak of 1418 kN at [e] 4.26s. This is actually smaller than the shear resistance reported in both Section I and II (1613 kN and 2062 kN). However, with further development of the diagonal cracks, the lateral load dropped to 1224 kN and the displacement reached its temporary peak, at [f] 4.33s. This lateral load was about 86% of the former peak value at [e] 4.26s, suggesting the resistance loss probably due to the noticeable occurrence of horizontal and diagonal cracks. In following peaks (e.g. [g] 5.37s), the lateral load could not exceed the peak lateral load at [e] 4.26s. Furthermore, after [h] 6.60s, the response displacement became much greater with smaller lateral load, because of great failure such as crushing of concrete at upper cut-off. Thus, [f] 4.33s is suitable to be considered as the start point when ultimate stage was reached, since there is an obvious decrease of lateral load.

4. EVALUATION ON FAILURE MECHANISMS AND SHEAR RESISTANCE

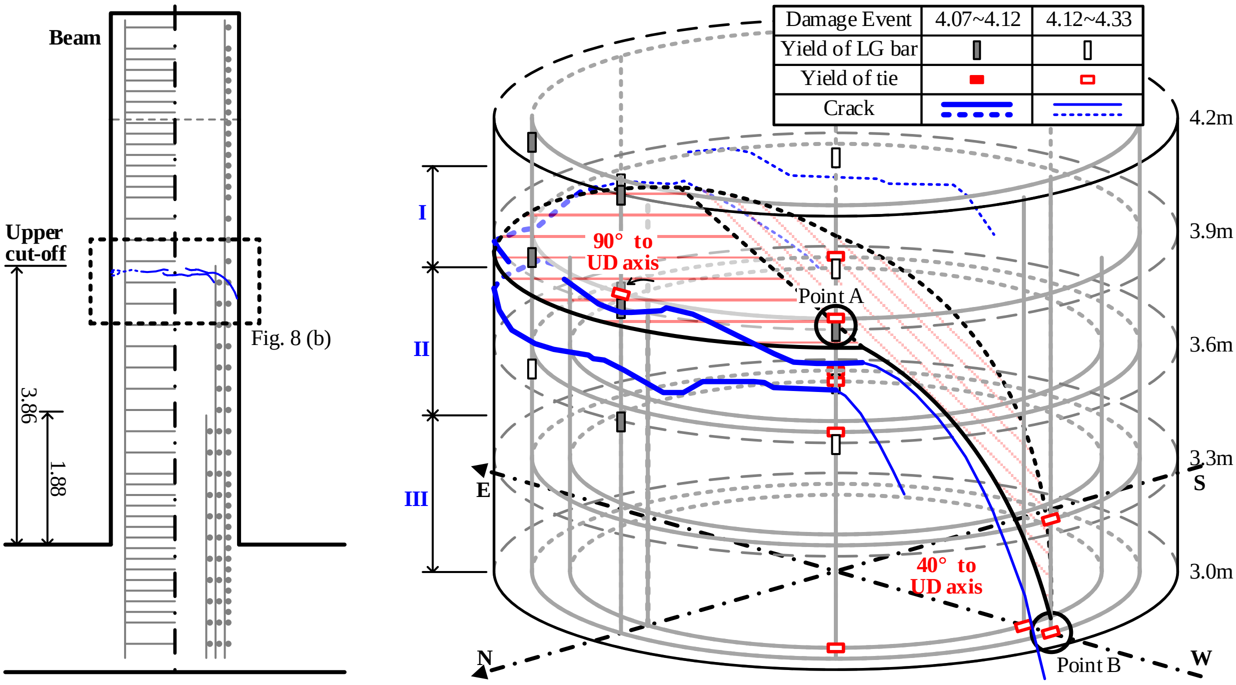
The general response of C1-2 in experiment was stated in Chapter 3. In this chapter, for verifying shear resistance and its mechanisms, the detailed response, as strain of LG bars and hoops, their relationship with cracks, and the failure progress, are evaluated.

4.1 Detailed Damage and Shear Failure Surface

As already mentioned, LG bars of the middle layer were cut off at the height of 3.86m, which is called the upper cut-off position. Around this position, the strain of LG bars and hoops was measured at height of 3.90m, which is actually higher than the just point of cut-off by about 1.4Φ as shown in Fig. 7 (a). Below this just point of cut-off position, the strain of LG bars and hoops was measured at actual height of 3.60m (-9.0Φ), 3.30m (-19.3Φ), and 3.00m (-29.7Φ), as shown in Fig. 7 (a). To evaluate different strain value at different height around the upper cut-off position, the response histories of average strain in LG bars (in Fig. 7 (b)) and hoops (in Fig. 7 (c)) are plotted and compared. Here, the average strain at each height is defined as the average value of total 8 strain gauges attached on LG bars or hoops at a particular height. Thus, this average strain is suitable to explain the general damage development at each height, rather than the strain of any single strain gauge to explain the local damage condition.

The average strain history shown in Fig.7 (b) shows that the value measured at height of 3.90m (1.4Φ) increases rapidly from [c] 4.07s (first visible horizontal crack) and exceeds the yield strain of 1850μ, probably due to flexural response. Then, it continues to raise and reaches over 8000μ at [f] 4.33s (when diagonal cracks firstly occurred). After [g] 5.37 s, the strain at this height becomes negative, suggesting that the LG bars may have buckled. However, the strain at height of 3.60m (-9.0Φ) only slightly exceeds the yield strain once and does not reach to notable value until [g] 5.37s. Besides, the average strain on the other two heights does not develop significantly. As a result, it can be inferred that the flexural response of column firstly affects the LG bars near the just point of cut-off position, and this influence extends to lower sections with further damage.

On the other hand, as the average strain of hoops illustrated in Fig. 7 (c), the strain develops significantly



(a) General crack at 4.33 (from N)　　(b) Detailed damage of LG bars, ties and cracks

Fig. 8 Development of damage event (4.07~4.33 sec, at when shear crack occurred firstly)

at height of 3.60m (-9.0Φ) from [c] 4.07s to [f] 4.33s, beyond the yield strain (2050μ) of hoops. Additionally, the strain at two lower heights, 3.30m (-19.3Φ) and 3.00m (-29.7Φ), also reaches yield strain. This notable response strain at these heights lower than the cut-off position is caused by the diagonal shear cracks occurred at [f] 4.33s. Then, the average strain in hoops at height of 3.90m (+1.4Φ) increases noticeably beyond yield and reaches similar level of that at lower heights by [g] 5.37s. Considering the negative strain of LG bars at this height (seen from Fig. 7 (b)), the possible buckling of LG bars can be considered as the main cause of damage of hoops.

To further evaluate the failure phenomenon and the response strain, and their interactive relationship, Fig 8 was plotted. Fig. 8 (a) shows the general position, and (b) shows the detailed crack condition in detail, and the yield of LG bars and hoops, during the time period when horizontal cracks and diagonal cracks occurred. Here, to explain the progress of failure, this time period is separately evaluated for the occurrence of the horizontal cracks (from [c] 4.07s to [d] 4.12s, and marked by solid rectangular and thick lines) and for the occurrence of the diagonal cracks (from [d] 4.12s to [f] 4.33s, marked by hollow rectangular and the thin lines).

For the first period (from [c] 4.07s to [d] 4.12s), the cracks occurred in the surfaces of column from E side to NW side at the height of about 3.80~3.90m. Besides, 4 of total 8 LG bars (the 1st layer without cut-off) at height of 3.9m reached yield, as well as another 2 at 4.2m and only 1 at 3.60m. Therefore, according to the yield condition of the LG bars, it is reasonable to assume that internal crack may develop until half of the section at 3.90m. Then, from [d] 4.12s to [f] 4.33s, the formerly occurred horizontal cracks extended diagonally downwards from NW side to W side until height of about 3.0m, and from E side to almost S side. In addition, hoops began to yield at several places, such as 3.60m and 3.90m in NW side and 3.0m and 3.30m in W side, simultaneously. Thus, based on the measured result of strain gauge on hoops, the internal failure surface can be assumed and the angles to the vertical axis can be obtained as about 40°, by connecting Point A (3.9m, NW) and Point B (3.0m, W) shown in Fig. 8 (b).

4.2 Evaluation on Shear Resistance

Based on the explanation in former section, it can be concluded for C1-2 specimen, the shear failure occurred at the upper cut-off position after the yield of several LG bars and the horizontal cracks until half of section. After this horizontal crack (about 90°) due to flexure, as shown in Fig. 9 (b), the shear resistance degraded and the diagonal crack (about 40°) occurred. As a result, by connecting the start point and the end point of the actual cracks, the actual shear angle was about 60° to the vertical axis. This suggests that the horizontal crack due to flexure caused the larger shear angle of 60°, compared to the standard assumption of 45° shear surface based on specification. This further reduced area of concrete and number of hoops to resist the shear load. Thus, the shear resistance (V'_R) by the actual failure surface can be expressed as:

$$V'_R = \tau_c A_c \cot\theta + (f_y A_h / s) d \cot\theta \qquad (3)$$

where τ_c: shear strength by Kawano[6] (=0.76 MPa, with LG bar ratio of 1.16%); A_c: area of cross-section along cracks; θ: angle between shear crack and vertical axis (refer to Fig. 9 (a)); A_h: area of hoops in shear failure section; s: distance between hoops; d: height of section.

The newly calculated shear resistance based on the actual failure surface can be calculated $V'_{R\text{-}II}$ = 1270 kN, as plotted in Fig. 9 (b) for the section below the just point of upper cut-off position (Section II). As a result, this shear resistance ($V'_{R\text{-}II}$ = 1270 kN), due to reduced area of concrete and number of hoops to resist the shear load, is smaller than the initial shear resistance ($V_{R\text{-}II}$ = 2062 kN) by 38.4% and smaller than the flexural resistance ($F_{y\text{-}II}$ = 1369 kN) by 7.2%. Besides, this shear resistance coincide with the [f] 1224 kN (4.33s) when ultimate stage was firstly reached, stated in Section 3.2 (Fig. 6).

Besides, the reduction of shear resistance with the increase of shear angle is plotted in Fig. 10. It can be found that concrete and hoops provide similar shear resistance and decrease similarly. Therefore, the total shear resistance decreased from $V_{R\text{-}II}$ = 2062 kN with 45° assumption according to specification to $V'_{R\text{-}II}$ = 1270 kN with 60° in actual failure.

To sum it up, due to the flexural failure at upper cut-off position, horizontal cracks developed up to half of section and reduced concrete area and hoop number to resist shear because of the greater angle (60°) to vertical axis. This caused the range below the upper cut-off (Section II) being the critical section with reduced shear resistance ($V'_{R\text{-}II}$ = 1270 kN) by 38.4%.

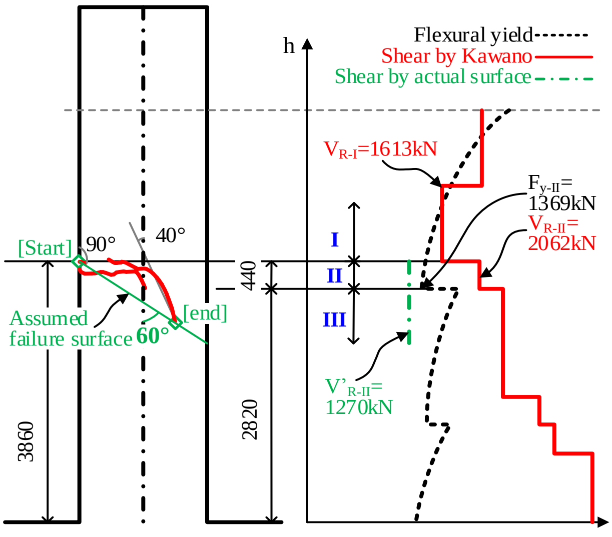
5. CONCLUSIONS

Based on the experimental results of the RC column with cut-off LG bars based on E-Defense excitation, the detailed response displacement, strain of LG bars and hoops, the development of damage, and their interactive relationship are discussed in details. The following conclusions have been drawn:

(1) In the experiment of C1-2, cracks firstly occurred horizontally at the upper cut-off point probably due to flexural response, and then extended into diagonal cracks due to shear. This resulted in the reduction of the lateral resistance of the column.

(2) According to the detailed evaluation of the failure position, it was found that, at the just point of the upper cut-off position, the LG bar suffered the most significant and earliest failure, such as flexural yield and even buckling. On the other hand, at lower sections, as the height considering the development length (about 15.2Φ for C1-2), hoops suffered yield but not noticeable failure, after the occurrence of flexural cracks. This indicated the just point of the cut-off position should be considered as the crucial section that led to the further severe failure.

(3) Based on the assessment on the failure mechanisms and the shear resistance, it was found that due to early horizontal crack until half of section by flexure, and following diagonal crack by shear, the actual shear angle (60°) is greater (than assumed 45° in specification), below the just point of upper cut-off point. Thus, with smaller concrete area and less hoops to resist shear load, the total shear resistance was reduced greatly by 38.4% and severe shear failure occurred to this height.



(a) Reinforcement setup (b) Resistance

Fig. 9 Shear Resistance based on Actual Failure Surface

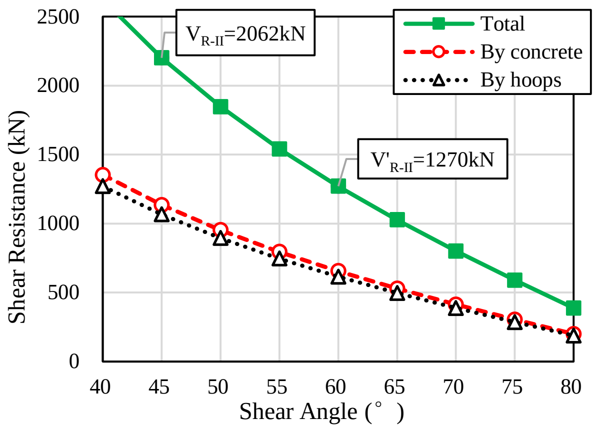


Fig. 10 Influence by Shear Angle on Shear Resistance

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