A PRELIMINARY STUDY ON THE BOND FAILURE OF LARGE-SIZE REBAR IN BEAM DURING BENDING TEST USING ACOUSTIC EMISSION MONITORY

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ABSTRACT

Reinforcing bars of size #14 and #18 were used generously in the nuclear power plants (NPP) constructed near seismic belt or fault area. A great number of these steel bars were matted and embedded within the mass concrete of containment building to withstand earthquake. However, few researches and experiments could be the references to the bar bonding and anchorage evaluation of these sizes. After a series bonding tests for previous study, this research aimed to investigate the bond behavior and failure mechanism of large-size rebar within structure element. To begin with, reinforced concrete (RC) beam under flexural load was selected. Three beams, reinforced with single #10, #14 and #18 tension rebar respectively, were tested conforming to the ASTM C78. During the test, the acoustic emissions (AE) due to cracking, shearing, rubbing, de-bonding or interlocking were monitored and recorded using a real-time high sampling rate DAQ system and proper sensors; besides, the displacement, cracking situation, failure mode, yield load and ultimate load of each beam were measured routinely.

In this paper, the existence of de-bonding and the yielding behavior of beam were addressed, especially the major bond failure occurring in the post-yield period. The track between bond fracture development and the final flexural or shearing failure could be exhibited and chronicled through its AE performance. It was also concluded that the early bond failure of #18 and #14 rebar decreased the beam ductility after yielding. Limited safety margin was allowed for the large-rebar structure becoming nonlinear.

INTRODUCTION

Bounding Problems in Large-size reinforcement

Large-size reinforcement in RC was mostly applied to solidify the massive containment structure of nuclear power plant (NPP) for anti-seismic reason. In the NPP constructed near seismic belt or fault area, a great number of the #14/#18 rebar mats were embedded within the RC bearing wall, foundation plate and floor slab. However, #14 and #18 steel bars were rarely used in common construction, not only for their high cost and poor workability, but also the suspect in their bonding performance. Few previous researches and experiments could be referred to for evaluating the bar bonding of these sizes. The influences of bar size on bonding, property matching and performance of RC were not fully exposed.

The basic principle of RC mechanism is that sufficient boundary conditions must exist between the steel reinforcement and the concrete to transfer the acting force between the two; and this transference is provided by the adhesive force at the rebar-concrete interfaces. This acting force was called bonding; the bond stress was defined as stress transferred along the boundary. In previous studies, Chinn, Ferguson and Thompson (1955) conducted beam displacement loading tests on general concrete to explore the effects of protection cover and the failure model during de-bonding. Later, Ferguson et al. (1965) used the pullout test to research bond strength. The #14 and #18 reinforcing bars, wound around with spiral hoops,
were tested too. Their results indicated that specimens with longer embedded rebar achieved higher bar stress and bond strength. The amount of sliding at the force-bearing side generally had a proportional correlation to the steel bar diameter. It was noted that the embedded spiral hoops could restrain the protection concrete from split failure in their pullout tests. However, most #14/#18 steel bars embedded within NPP concrete were not with spiral hoops. The split failure on #14/#18 rebar was not fairly studied.

ACI 408 Committee (1966) discussed the nature of bond failures and investigated the effects of cracks. The factors causing de-bonding cracks were related to the weak surface and the development of bond stress, and determined by the beam width and rebar spacing or gaps. Two failure models, pullout failure and split failure, occurred commonly during the bonding test. When the protection cover of rebar was sufficiently thick, the loaded concrete tended to break or to shear along the bar between the ribs. Bond failure resulted in the rebar being pulled out of the concrete, causing the concrete to have a hole instead of splitting cracks. In the second model, when the protection cover was insufficient and no spiral hoop was wound around, the concrete surrounding rebar would be split due to the pushing by ribs. The splitting cracks would damage the bonding interfaces.

Most researches on RC bonding test focused on the rebar of common size (#6~#10) under a general protection cover. To approach the RC design for NPP, the bond behaviors of rebar in various sizes of #18, #14, #10 and #6 were examined through a series of pullout tests following to ASTM C234 in the previous study of Kan and Pei (2013). In their research, three concrete mixtures of strengths $f'_c$ (4000, 5000 and 6000psi) and 4~5″ concrete protection cover were adopted to make the specimens. No spiral hoop was embedded to enhance the bond of concrete. Therefore, the in-site de-bonding mechanism could be approached. The test results indicated that the maximum tensile stress in the #18 rebar was approximate 12% to 16% of the yield strength ($f_y$) when the specimen reached the ultimate strength ($f_u$), just before split failure. For the concrete with #16 rebar, the maximum tensile stress in rebar was about 19% to 24% of $f_y$, when the ultimate load was applied. The 4~5″ concrete cover could offer an maximum bonding force to tension the #10 rebar to 34% of $f_y$, and almost to yield the #6 rebar. In consequence, large-size rebar could not dominant the ultimate bond strength in RC; de-bonding damage of the #18/#14 rebar without spiral hoops or other preventer could cause early or suddenly failure in RC structure.

**Acoustic Emission Monitoring**

When the RC composite was fractured, acoustic emissions (AE) of many frequencies would perform. Usually, the sound coming with the final burst of RC was loud and familiar, whose frequency was surely under 20kHz and available for human. However, most AE events, due to slightly cracking, shearing, rubbing, de-bonding, interlocking, etc., were ultrasounds, certainly out of human hearing range. Different event cause might create signal with different characteristic and frequency. Concrete was noted for a high-decay-rate material for ultrasound. After the filtering out through the low-pass-filter medium, most receivable and identifiable AE were with main frequency under 150kHz (2007, 2008). To record and identify the signals, high sampling rate DAQ system and proper sensors were required.

Like the previous studies of Pei et al. (2007, 2008, 2009, 2010), a suitable NI based system with 4-digital-channel/800kHz/16bits DAQ was used for this feature, AE received by the preamplifier sensors could be digitized through the system and recorded as oscillatory signals. Signals from different sensors on the specimen could be real-time monitored and presented on screen during the loading test; the recorded data could be replayed and analyzed after the test.

In this research, four sensors were attached to the beam specimen at certain locations, where the bending or shearing cracks could occur nearby. After the bending crack initiated on beam surface, the fracture developments within the beam concrete were the interests to be followed, especially the inner de-bonding fractures. By threshold filtering, the signal oscillations could be counted by second to create AE signal density (hit count/second) record over the testing period for each sensor. The obtained AE densities and loading/displacement record would be graphed in a historical chart. The fracture mechanism of the loaded specimen and the de-bonding performance of the large-size rebar could be revealed by analyzing the chart.
Load Test

This research aimed to investigate the bond behavior and failure mechanism of large-size rebar within structure element. Not like the cube specimen used in the previous pullout test (ASTM C234), the bond condition between rebar and concrete in structure could be changed or enhanced due to the surrounding restraint by structure shape, stirrups and loading stress. Therefore, not only the basic failure models, pullout and split, appeared; mixture performance of de-bonding and yielding of the large rebar were expected. To begin with, RC beam under flexural load was selected to plan this experiment. Three beams, reinforced with single #10, #14 and #18 tension rebar respectively, were tested conforming to the ASTM C78. During the test, real time AE monitoring and recording were used to detect the cracking, shearing, rubbing, de-bonding or interlocking in the beam; besides, the displacement, cracking development, failure mode, yield load and ultimate load of each beam were measured routinely. The track between the de-bonding fracture development and the final failure would be chronicled and revealed.

EXPERIMENT DESIGN

Three beams for the four-point bending tests were 0.65m(h)×0.35m(b)×5m(L) in dimension. The designed strength of beam concrete $f'_c$ was 410 kgf/cm$^2$, and the strength of rebar $f_y$ was 4219 kgf/cm$^2$ (ASTM A615 GR60 RO). To assure the beams failed by bending or shearing, and to conduct the de-bonding fracture in tension reinforcement, a design of under-reinforced section in bending with both tension and compression reinforcement was followed. In Figure 1, three or four #10 bars, framed by stirrups (shear reinforcement), were used to enhance the compressive strength of beam. The enhanced sections were to prevent early compression failure on beam, and to direct the beam yielding. In this research, the beams numbered as B-S10, B-S14 and B-S18 were embedded with #10, #14 and #18 single rebar as tension reinforcement respectively. The protection cover of tension rebar was designed over 10cm in concrete, just like the NPP requirement. The sufficiency of cover thickness for different bar-size would be reviewed after the tests.

In the load test, MTS applied bending force through a steel spreader beam and two roller-joints on the beam specimen (Figure 2). The loading speed was controlled by a preset time-displacement ratio; therefore, the bending deformation and fracture could develop linearly with time. During the test, the loading would be halted several times to check/contour the beam surfaces. The occasions were called due to the AE performance in real-time monitoring.

During the test, two LVDTs settled under the beam center were to measure the vertical displacement; the obtained displacement-loading data could indicate the mechanical property of each beam. In Figure 2, positions of the four AE sensors are pointed. Sensor #1 and #3 were fastened on different side-surfaces near the 1/4 and 3/4 span of the beam respectively, where the shearing fractures generally happened. Sensor #2 and #4, located on different side-surfaces but near the beam center, were to record the acoustics especially due to the early bending cracks or the late crushing under loading joints. The distance between sensor #2 and #4 was about 0.7m. Senor #2 and #3 were located on the same side with an interval of 1.55m; #1 and #4 were on the other side. Because of the space between sensors and the decay-rate of concrete, AE signals received by sensor #1, #2(#4) and #3 could be localized. Bending crack and shearing fracture could be classified according to the receiving sensor and received time.

RESULTS OF THE LOAD TESTS

Load Testing of Beam B-S10

The beam was gradually bended till failure with an ultimate load ($P_u$) of 407.07kN and a displacement ($\Delta u$) of 78.33mm. To close exam the beam surfaces, the loading process had been stopped at 122, 147, 177, 294 and 333kN. The halt at 364kN was specially to adjust the loading rate. In Figure 3, two photos present the developed cracks and the in-site measurement on beam surfaces. Figure 4 shows
the historical chart of AE densities, loading and displacement of the test. In this chart, the halts for examination and adjustment were skipped because of the acoustic silent in these periods. Fracture rarely progressed when the loading was halted or decreased. According to the known material property and required serviceability, the designed load of this beam (P_d) was 164.90kN. After test, the yielding load (P_y) and displacement \( \Delta_y \) were indicated as 291.21kN and 8.55mm respectively. Where the \( P_n = 0.57P_y = 0.41P_u \) could be referred to evaluate the real flexural crack control. Beam B-S10 was finally broken by bending failure; the major damage occurred near the beam center.

According to the AE monitoring result, the loading at initial cracking (P_cr) was 117.36kN. Four bending cracks, which were spaced 0.4m~0.6m apart horizontally, extended upward over the beam midline when the load was over 147kN. The measurement in Figure 3 and the AE density in Figure 4 revealed that the bending cracks extended fast at the loading stage from 122kN to 232kN. During the loading from 177kN to 294kN, the measurement indicated that the surface cracks were dormant at the neutral axis, but the AE densities recorded by sensor #4 and #2 were quite active. Since the beam was continually deflected at the same rate, the crack-openings had to keep enlarging and elongating the tension rebar. Acoustics could be created for this mechanism. AE density from sensor #3 exploded and diminished during the loading period of 240kN~278kN. Minor shearing fracture might occur for a while. Beam yielding began at 291kN. During the post-yield period from 291kN to 407kN, bending cracks stopped growing upward but kept enlarging; some cracks turned horizontally for the compression effect.

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Figure 1. Design of the beam specimen and the three sections of reinforcement

Figure 2. The four-point bending tests (ASTM C78) and the sensor locations
On the bottom surface, most cracks crossing the beam were recorded before the loading of 147kN. But, the cracks parallel to the tension rebar were observed when the load was over 294kN. The later fractures were caused by the de-bonding of rebar and extended out through the concrete protection cover. In Figure 4, the enlarged chart indicated that the displacement rubbing between rebar and concrete, or the pullouts, caused the bumpy loading line and fluffy AE densities. Because of the controlled loading rate, the above pullouts appeared rhythmically. In Figure 3, the de-bonding cracks distributed on the middle part of beam, and did not extend out of the bending area between the loading roller-joints. The anchorage bond of the tension rebar still held during the post-yield period.

Figure 3. Crack development on B-S10 beam (photos taken after the bending test)

Figure 4. AE densities, loading and displacement historical chart of the load test on B-S10
Load Testing of Beam B-S14

In this test, beam B-S14 was gradually bended till failure with an ultimate load \( (P_u) \) of 615.17kN and a displacement \( (\Delta_u) \) of 71.55mm. The loading process had been stopped at 119, 147, 177 and 294kN to check the beam surfaces, and halted at 543kN to adjust the loading rate. According to the material property and design factors, the designed load of beam \( (P_n) \) was 284.63kN. The yielding load \( P_y \) and displacement \( \Delta_y \) were 496.02kN and 14.14mm respectively. Hence, \( P_n = 0.57P_y = 0.46P_u \) was noted. In Figure 5, two photos present the crack development on beam. Two or three major cracks conducted the final bending failure. Figure 6 shows the historical chart of AE densities, loading and displacement of the test, which was modified as the previous case.

The AE monitoring result indicated that the loading of initial crack \( (P_{cr}) \) was 108.17kN. Later, five bending cracks, spaced 0.4m~0.6m apart horizontally, extended over the beam midline when the load was over 177kN. The in-site measurements and the rising AE densities of sensor #2, #3 and #4 showed that the major bending cracks developed during the loading from 119kN to 294kN. After 177kN, the crack extension became slow and crooked; but the shearing actions at the areas under the loading roller-joints were noticeable. In Figure 6, the major shearing fractures occurred during the loading from 221kN to 516kN. The density peak of sensor #3 due to initial shearing burst was noted at the loading of 224kN; the peak of sensor #1 was at 421kN. The shearing cracks on the left beam started earlier, and seemed more complicated than the cracks on the right beam. After the yielding, shearing action was shut off.

During the post-yield period from 496kN to 615kN, some bending cracks might extend horizontally due to compression. Two or three new bending cracks were observed. According to the AE results, they might happen at/after the halt period of 543kN. The loading rate adjustment speeded the beam deformation, but might also resettle the balance on spreader beam to cause new cracks.

On the bottom surface, most bending cracks were recorded before the loading of 147kN, however, most longitudinal cracks developed after yielding. The later fractures could be caused by the de-bonding on #14 rebar. The enlarged chart of Figure 6 indicated that the displacement rubbing between #14 rebar and concrete caused the bumpy loading line and the spiculate AE density. Because of the fixed loading rate, the above rubbing and AE spicules performed rhythmically. In Figure 5, the de-bonding splits began from the bending area; extended longitudinally and approached the beam supports. The anchorage bond of #14 rebar was certainly damaged during the post-yield period, especially after 1600sec. (599kN). The pullouts in tension rebar allowed the beam more deflection but flattened or weakened the beam strength. The beam was finally broken by bending failure at 2017sec. (615kN).

Load Testing of Beam B-S18

In this test, beam B-S18 was gradually bended till failure with an ultimate load \( (P_u) \) of 911.12kN and a displacement \( (\Delta_u) \) of 40.68mm. The loading process had been stopped at 120, 147, 177, 294 and 836kN to check the beam surfaces, and halted at 884kN to adjust the loading rate. The designed load of beam \( (P_n) \) was calculated as 493.52kN. The obtained yielding load \( P_y \) and displacement \( \Delta_y \) were 799.78kN and 16.46mm respectively. Where the \( P_n = 0.62P_y = 0.54P_u \), higher than the previous case. In Figure 7, two photos present the complicated crack development on the beam surfaces. Major shearing fractures and de-bonding cracks conducted the shear-tension failure on beam B-S18. Figure 8 shows the historical chart of AE densities and mechanical performance of the test. The halt periods were modified as the previous cases. In this chart, the loading of crack initiation \( (P_{cr}) \) was shown at 98.92kN.

In this case, four major bending cracks, spaced 0.5m~0.7m apart horizontally, extended upward over the beam midline when the load was over 177kN. The measurements on beam surfaces and the rising AE density of sensor #4 disclosed that most developments of bending crack occurred during the loading from 120kN to 294kN. After the halt at 294kN, shearing actions developed instead. In the chart, the rising AE density peaks of sensor #1 and #3, caused by shear cracks initiating on the right and left beam, were noted at the loading of 265kN and 288kN respectively. Because of the different local condition, the shear
cracking on the right beam seemed early-developed and more complicated. More than two inclined cracks kept growing till the surface checking at 836kN; the crack openings kept enlarging till the failure. Since the early cracks might block sensor #1 from acoustics, the detectable shearing actions on the right beam halted at 784kN and diminished after the yielding. The detection of sensor #3 performed well; the fracture on the left beam could be traced till the end of test.

During the post-yield period from 800kN to 911kN (989~1719sec.), most shearing actions moved to the regions near the roller-joints of loading. During 1238~1244sec. (886kN), density peaks of sensor #2 and #4 appeared; some crushing occurred under the roller-joints. At the same time, AE densities of sensor #1 and #3 dropped obviously; shear cracking was paused.

Figure 5. Crack development on B-S14 beam (photos taken after bending test)

Figure 6. AE densities, loading and displacement historical chart of the load test on B-S14
On the bottom surface, bending cracks along the beam cross-section were found mostly before the loading of 294kN; the longitudinal cracks were noted after the beam yielding. We noted that the loading stress normal to the tension rebar along the bottom surface was usually compressive because of the Poisson’s effect. Therefore, the de-bonding cracks on the bottom surface could be restrained to some extent. On the side surfaces, anchorage failure due to the shear tension occurred after 1450sec. (895kN). The de-bonding cracks horizontally distributed from the beam supports to the shear cracks. Not like the previous cases, the spiculate AE density and the bumpy loading line due to pullouts did not clearly appear in Figure 8. The vastly rising of AE, received by sensor #3 and #4 due to the shear tension failure at the period of 1525–1719sec., appeared instead. The anchorage bond was severely damaged for the large crack opening. Fewer pullouts occurred before the beam was broken.

![Figure 7. Crack development on B-S18 beam (photos taken after bending test)](image)

![Figure 8. AE densities, loading and displacement historical chart of the load test on B-S18](image)
DISCUSSION AND CONCLUSION

In above tests, the MTS loading speed was controlled by fixed displacement rate. The recorded vertical displacement and the corresponding load of each beam test could be graphically represented as the load-displacement relation in Figure 9. Since all three beams were yielded by MTS; the tension re-bars of #10, #14 and #18 were certainly yielded in the tests. The different rebar section areas, #10−814mm²; #14−1452mm²; #18−2579mm² (about 1:1.78:3.17), resulted in different strengths and stiffness of beam B-S10, B-S14 and B-S18 (see Table 1). The \( P_{y} \)s of B-S10, B-S14 and B-S18 were 291.21, 496.02 and 799.78kN, respectively (about 1:1.70:2.75); the \( P_{ys} \), 407.07, 615.17 and 911.2kN (about 1:1.51:2.24). The beam stiffness can be simply evaluated by checking the slope of load-displacement curve before yielding, or the \( P/\triangle y \) value. The obtained slope of B-S10, B-S14 and B-S18 were 34.06, 35.08 and 48.59kN/mm. The amount of tension reinforcement, \( P_y \), \( P_o \) and the stiffness of Beam B-S18 were the highest, however, the beam strength was not fairly proportional to the rebar section area.

The toughness, or ductility, of beam can be described by the beam performance after yielding; usually can be evaluated using \( \triangle u/\triangle y \) value. The obtained \( \triangle u/\triangle y \) values of B-S10, B-S14 and B-S18, which were 9.16, 5.06 and 2.47 respectively, indicated that the beam with large-size rebar performed poorly in the beam ductility. In this study, de-bonding damage and anchorage failure mainly conducted the beam behaviors of B-S14 and B-S18 after yielding, especially shortened the \( \triangle u \) (see Figure 9). The early bond failure of #18 and #14 rebar decreased the beam ductility after yielding, as well as the safety margin of large-rebar structure in nonlinear situation.

The loading period from crack initiation (\( P_o \)) to the dormant stage of bending cracks can reveal the influence of tension rebar on crack control. The periods on beam B-S10, B-S14 and B-S18 were at 29%-57%\( P_o \), 18%-48%\( P_o \) and 10%-32%\( P_o \), respectively. Larger rebar performed better control on crack extension and enlargement. The designed load capacities, \( P_{ys} \), of beam B-S10, B-S14 and B-S18 were 41%\( P_o \), 46%\( P_o \) and 54%\( P_o \). By comparison with the designed \( P_y/P_o \), the crack control of beam B-S18 was more conservative in the real test.

According to the obtained AE density charts, we can illustrate the loading periods in which major part of shearing fracture developed. The periods on beam B-S10, B-S14 and B-S18 were at 59%-68%\( P_o \), 36%-84%\( P_o \) and 29%-97%\( P_o \), respectively. The longitudinal cracks on the bottom surfaces of beam mostly developed after the yielding. The de-bonding due to large rebar elongation might initiate these splits. The later rebar pullouts on beam B-S10, B-S14 and B-S18, due to anchorage bond loss or failure, appeared at the loading over 343kN (0.84\( P_y \)/1.18\( P_o \)), 599kN (0.97\( P_y \)/1.21\( P_o \)) and 895kN (0.98\( P_y \)/1.12\( P_o \)), respectively. These rates disclosed that shearing fracture and anchorage loss on beam B-S14 and B-S18 closely related to the failure mechanism. The coupled fracture broke the beam B-S18.

In the test of beam B-S18, the 15cm concrete protection cover with stirrups could not restrain the splits on the side surfaces during the anchorage bond loss. De-bonding can be improved or prevented in recent constructions. Rebar anchors or spirals are used to enhance the anchorage bond of large-size rebar. However, most #14/#18 steel bars used in the old NPP RC structures were not fortified with the above accessories. Since the massive RC structures of NPP were designed to maintain material/structure linearity in most conditions, the anchorage bond loss/failure of #14/#18 rebar after yielding seemed far from happening in NPP. After Fukushima Daiichi nuclear disaster, structure fortification was considered in many old NPP located near the seismic belt or fault area. Since the safety design basis of NPP has to be advanced for new geological evidence or public requirement, the non-linear behavior of large-size rebar, including the anchorage bond loss and property matching, should be reviewed before the fortification.

In this research, the analysis result of AE monitoring fairly revealed the time and trend of each fracture mechanism occurred during the test. The discovered bond behavior and failure progress can be referable to evaluate the RC structure with large-size bars mechanically or acoustically. AE technique was useful and will be used in the future exploration of the extreme loading or non-linear behavior of RC. After this study, more tests will be needed to confirm and quantify the bond behavior of #14/#18 rebar. Furthermore, the bond of large-size rebar mat can be a good topic to research.
Table 1. Results of the bending tests

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<th>Beam No.</th>
<th>( P_{cr} ) kN</th>
<th>( \triangle_{cr} ) mm</th>
<th>( P_y ) kN</th>
<th>( \triangle_{y} ) mm</th>
<th>( P_u ) kN</th>
<th>( \triangle_{u} ) mm</th>
<th>( \triangle_{u}/\triangle_{y} )</th>
<th>( P_{cr}/P_u )</th>
<th>( P_y/\triangle_{y} )</th>
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REFERENCES