Behavior of RC Deep Girders that Support Walls with Short End Shear Spans

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Abstract

Two design approaches, conventional and strut-and-tie procedures, are developed for reinforced concrete continuous deep girders which transmit the gravity load from the upper wall to the lower columns. This paper presents the results of tests and analyses conducted on two specimens; the first specimen employs the conventional procedure, while the strut-and-tie procedure is used for the second specimen.

The conclusions are as follows: (1) The approach of the strut-and-tie method is valid for this type of continuous deep girder rather than the conventional beam approach. (2) Since the upper load is carried over directly to the supporting column through the stiff concrete strut to the point of yielding of the bottom ties, the shear capacity of continuous deep girders is mainly governed by the yielding forces of the bottom ties. (3) The additional shear resistance derives from continuity with the adjacent beams or walls. Shear and top reinforcements in the continuity region can be designed by using appropriate models for the additional margin of safety in terms of strength and ductility. (4) Simulation through two-dimensional nonlinear analyses using DIANA shows a good correlation with the experimental results.

Keywords: reinforced concrete; continuous deep girder; strut-and-tie method; DIANA

1. Introduction

The structural system for multi-purpose buildings in Korea is commonly a combination of a moment-resisting frame for the lower stories and a bearing-wall system for the upper stories. The lower stories usually accommodate parking areas, commercial spaces, gardens, or open spaces for architectural purposes, while the upper stories are generally used for residential apartments. In this type of building structure, deep girders transmit the load from the upper bearing wall to the lower frame. Girders that support walls that are set back from the column lines are generally deep and the principle of Bernoulli ("A plane section remains a plane after deformation") does not apply. Nevertheless, practicing structural engineers apply this principle in the design of these girders. This convention is uneconomical and causes difficulties in construction due to the excessive depth of the beams and the congested reinforcement. Moreover, the resulting design may not necessarily ensure safety and efficiency in structural behavior. In spite of this strut-and-tie modeling has recently been used as an alternative procedure in the Appendix of ACI 318-05(1), this method still needs more experimental and analytical elaboration for practical application.

Over the past several decades, many researchers have conducted experiments to increase the capacity of reinforced concrete deep beams, and have proposed new design equations for effectively predicting or estimating the capacity and the mode of failure of deep beams(2,3,4,5). Since the proposal by Schlaich, Schafer and Jennewein(6) of the strut-and-tie procedure for the design of stress-disturbed members or regions, many researchers have applied this model to deep beams for developing design methods or equations, and have verified their proposals by comparing them with test results(7,8,9). MacGregor(10) showed that the ductile behavior of continuous deep beams can be ensured by the use of strut-and-tie models. Recently, direct strut-and-tie model accounting for stress-distribution factors was proposed and verified by experiment(11), and a new model which takes into account the effect of the support stiffness in continuous deep girders was suggested(12). However, the adopted example was a two-span deep girder that transmits the load from the upper columns to the off-set lower columns. When deep girders transmit the load from the upper walls to the lower columns, the situation can differ from MacGregor's case because the upper wall can participate in the transmission of, as well as the resistance to, the gravity load. Kuang(13) investigated the failure mechanism and structural behavior of a 2 span transfer girder supporting in-plane loaded shear
wells. When the transfer beams support a full-span shear wall, they act as tension members. But, for half span shear walls, transfer beams act as a tension-flexure member. The objective of this paper is to clarify the behavioral characteristics of the reinforced concrete deep girders, which transmit the gravity load from the upper wall to the lower columns with a short end shear span. For the case of a long end shear span, refer to reference No. 14. To achieve the above objective, a 17-story reinforced concrete structure was selected as a prototype. The continuous deep girder between the upper wall system and the lower frame is designed according to the design procedure of ACI 318-95 recommendations. Experiments were conducted to investigate the behavior of specimens, and an analytical study was performed using the nonlinear-analysis program DIANA(13) to overcome the limitation of experimental results.

2. Prototype Building and the Design of Transfer Girders

Fig.1. shows the prototype of a multi-purpose building structure. The upper 15 stories are constructed as a bearing-wall structure, and the lower two stories consist of a space frame. The continuous deep girder transmits the gravity load from the upper bearing wall to the lower columns and the set-back of the upper wall from the column line usually induces a local disturbance to the force flow. The design strength of the concrete and the reinforcement are 30MPa and 400MPa, respectively.

2.1 Design by the conventional procedure

The transfer girder was modeled as a continuous beam. Fig.2.(a) shows design load from the upper stories for the prototype transfer girder. It was assumed that for this design the upper wall acts as a load rather than as a structural member. The concentrated loads represent the load from the orthogonal walls and girders. The transfer girder was analyzed on the assumption that it has a constant flexural rigidity throughout the length and that the supports by the columns are hinges. The member size, flexural reinforcement and the shear reinforcement were designed according to the design procedure of ACI 318-95(16). The total depth was determined to be 2150 mm. The detailed information on the flexural and shear reinforcements is given in Table 1. A comparison of the demand and supply in the flexural moment and the shear force is shown in Table 2.

2.2 Design by the strut-and-tie modeling procedure

By using the approach suggested by Schlaich(15), the struts and ties were modeled as follows: (1) elastic

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**Table 1. Comparison of Design Results**

| Location       | Top         | Bottom       | Horizontal reinforcement | Vertical reinforcement |
|----------------|-------------|--------------|--------------------------|------------------------|
| Top support    | 18-D25      | 7-D25        | 2-D19@250 (0.2%)         | 4-D13@90 (0.71%)       |
| Right support  | 14-D25      | 9-D25        | 2-D19@250 (0.2%)         | 4-D13@90 (0.35%)       |
| Center of span | 2-D25       | 12-D25       | 2-D19@250 (0.2%)         | 4-D13@180 (0.35%)      |

**Table 2. Required Strength and Design Strength (Conventional Approach)**

| Moment | $M_n$ (kNm) | $M_u$ (kNm) | $\varphi M_u$ | $\varphi M_u/M_n$ | $V_n$ (kN) | $V_u$ (kN) | $\varphi V_u/V_n$ |
|--------|-------------|-------------|--------------|-------------------|------------|------------|------------------|
| Positive | 4,620       | 5,630       | 5,067        | 1.07              | 693kN/m    | 3279kN     | 3.23             |
| Negative | 6,110       | 7,175       | 6,458        | 1.06              | 7437kN     | 2333kN     | 1.96             |

$M_n$: Moment due to factored load, $M_u$: Nominal moment strength, $V_n$: Nominal shear strength, $\varphi$: Strength reduction factor

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finite element analysis was performed to determine the general flow of forces from the upper wall to the transfer girder and finally to the columns; (2) using this information, the layout of struts and ties was configured; and (3) this model that consists of struts and ties was analyzed and the elements (struts, ties, and nodes) were designed for the applied forces.

To take into account the effect of the upper walls, a lower portion of the upper wall was included in the strut-and-tie model, as shown in Fig.3. The uniform gravity load in Fig.2.(c) was converted to the two equivalent concentrated loads at the quarter points. All the applied concentrated loads were increased by dividing them by the factor of \( \phi = 0.85 \). It was assumed that concentrated load from the orthogonal wall and girder was transmitted directly to support B through the compression strut. However, a portion of the load from the upper bearing wall was assumed to be transmitted through vertical ties in the shear critical zone. The force that was transferred through the shear reinforcement was purposely set to be 40% of the total shear force. The dimensions of struts were determined according to MacGregor\(^{10}\).

A comparison of the design results obtained from the two approaches is shown in Table 1. The adoption of the strut-and-tie approach allowed a reduction of the depth from 2150mm to 1800mm, as well as a reduction in the ratio of the vertical shear reinforcement from 0.71% to 0.35%.

3. Construction of Specimens and Experimental Setup

Considering the capacity of the laboratory, the prototype was reduced to a 1:2.5 scale. Even though the specimens were reduced to a 1:2.5 scale, the range of the load to be applied exceeded the capacity of the available loading system. Therefore, the width of the specimen had to be reduced by half. However, in this case, the width of the upper wall could prove to be too thin for carrying the applied concentrated loads without bearing failure to the transfer girder. Thus, the width of the upper wall was increased by two times. Since the yielding force of the bottom bars remains constant, the arm length between these bottom bars and the centroid of the upper compressive zone with increased thickness may increase a little. However, the increase of moment due to the increased wall thickness is estimated to be only 5%, and therefore negligible. Furthermore, due to the constraint on the length of the available loading frame, even the 1:2.5-scaled specimens had to be cut at the right-hand side of support B. To simulate the continuity of the prototype, the right end of the specimen was constrained with the lower and upper supports, B and C, as shown in Fig.4. Also, the uniformly distributed gravity loads in Fig.2.(c), or the equivalent concentrated loads in the strut-and-tie model in Fig.3., were further simplified by two concentrated loads that were at a constant ratio to each other in Fig.4. The reaction force at support C was measured by installing a load cell. Displacement transducers were installed to measure: the vertical displacement, \( \Delta_e \), at the mid-span; the displacement, \( \Delta_r \), at the left end-point of the shear span; and the displacement, \( \Delta_c \), at the bottom of the upper-right-hand support, C. Strain gauges were attached to the reinforcements for measuring the strains of the flexural and shear (vertical and horizontal) reinforcements at the critical locations. The view of the test set-up is given in Fig.5. Fig.6. shows the resulting details of the two 1:2.5-scaled specimens: AL-86 for the conventional approach and SL-72 for the strut-and-tie approach. The average concrete compressive strengths of specimens AL-86 and SL-72 were 43.2MPa and 38.2MPa, respectively, both of which were larger than the design strength (30MPa). The main reinforcements of the specimens were D10 bars, while the shear reinforcements were D5 bars, of which the average yield-strengths were 431.5MPa and 460.9MPa, respectively.

4. Test Results

4.1 Global behavior

Solid lines, Fig.7. show the relationship between the applied total load, \( P \), and the vertical displacement, \( \Delta_e \), at the mid-span for AL-86 and SL-72, respectively.
The ultimate strengths of specimens AL-86 and SL-72 were 1.5 and 1.08 times larger, respectively, than the required design load, $P_u$ (595kN), which is derived from the design load as shown in Fig.2. according to the similitude law (17).

The dashed lines in Figs.7.(a) and (b) are the relationship between the reaction force at support C and $\Delta_m$. In both specimens, the reaction force at support C was initiated approximately at the time of flexural yielding under the load point, P2, shown in Fig.7. Fig.8. shows the relationship between flexural moment under the load point, P2, in Fig.4., and the shear force at the left boundary of shear critical region denoted by gray zone in Fig.4. While the shear strength of AL-86 (537kN) exceeded the design shear strength, $V_u$ (392kN), that of SL-72 (382kN) slightly fell below the design strength. The characteristics of the behavior before and after this flexural yielding can be clearly noted in the interaction curve of the flexural moment under the load point, P2, versus the shear force in the shear span in Fig.8. Here, the flexural moment and the shear force proportionally increased up to the flexural yielding point. However, after flexural yielding, the shear force in the shear span kept increasing while the flexural moment remained almost constant. The bending moment strengths for the section at the load point, P2, and for the section wherein the existence of the upper wall is neglected as assumed in the conventional approach, are obtained by using the first principles in section analysis. The measured value of 369kNm of the yielding moment under the load point, P2, for specimen AL-86 is smaller than the value of 408kNm, as predicted by using the whole section, including the upper wall, and yet is about two times larger than 171kNm, which is the value obtained by neglecting the existence of the upper wall. In the same way, the measured yield moment of 245kNm at the load point, P2, for specimen SL-72 appears to be an intermediate value between that obtained by considering the whole section, 366kNm, and that predicted by using only the section of the girder, 122kNm. This implies that the conventional RC beam section analysis cannot be applied to this type of deep girder.

The overall relationship between the load and the mid-span deflection shown in Fig.7. can be idealized by the bilinear curve shown in Fig.9. These two lines are characterized by the behavior models of phase I and phase II, as shown in Figs.10.(a) and (b), respectively.
Table 3. Comparison of Measured $V_y$ and $M_y$ with those Predicted by Strut-and-tie Model

| Specimen | $V_y$ (kN) | $M_y$ (kNm) | $jd^*$ (mm) | $l^*$ (mm$^2$) | $\tan \theta^*$ | $A^*$ (mm$^2$) | $f_y$ (MPa) | $\Delta = A_s$ (kN) | $V_y = T \cdot \tan \theta$ | $M_y = T \cdot jd$ (kN.m) |
|----------|-----------|-------------|-------------|--------------|--------------|-------------|-------------|----------------|----------------|----------------|
| AL-86    | 388       | 369         | 1200       | 1765         | 1.569        | 499.1       | 431.5       | 251.9          | 337            | 259            |
| SL-72    | 263       | 245         | 1000       | 765          | 1.307        | 427.8       | 431.5       | 184.6          | 240            | 185            |

* For $jd$, $l$, $\theta$, and $A_s$, see Fig.10(a).

The behavior model of phase I is merely the truss model that consists of a direct strut that connects a certain point under the load-point, P2, the support, B, and the bottom horizontal tie. Therefore, the shear force, $V_y$, is entirely attributable to the concrete strut. The yielding moment, $M_{y_1}$, can be obtained by multiplying the yield force of the bottom tie, $A_s f_y$, by the arm length of this tie, $jd$, with respect to the center of rotation, the location of which can be approximated as the intersection of the vertical line through the load-point, P2 in Fig.4., and the diagonal line in the pure-shear span, as shown in the phase-I model in Fig.10. (a). The values of $V_y$ and $M_y$ that are predicted with this phase-I model (strut-and-tie model), using the assumed values of $jd$, $A_s f_y$, and $\theta$, are compared with the measured values in Table 3. It can be seen that though the predicted values provide lower-bound approximations to the experimental results, there exists significant discrepancy between the predicted and measured values. Since the values of the yield tie forces are relatively reliable, the main reason for the discrepancy is thought to be the difference between the assumed and actual values of $jd$ and $\theta$. Also, the contribution of the horizontal shear reinforcement to the yield bending moment might be another source of error. The reason for this discrepancy will be further discussed in the subsequent section of analytical simulation study.

In the behavior model of phase II, the reaction force, $R_c$, occurs for equilibrium with the additional load, $\Delta P$, after the flexural yielding and the additional shear, $\Delta V$, in the shear span is transmitted through the vertical shear reinforcements that cross the concrete strut. With further simplification of the phase-II model to a beam that has a plastic hinge, as shown in Fig.11.(a), the incremental shear force, $\Delta V$ (Fig.11.(b)), and the incremental flexural moment, $\Delta M$ (Fig.11.(c)), can be obtained by using the equilibrium relation, $R_c = 1.9 \Delta P$. $\Delta P$ and $\Delta V$ are shown as the differences between the solid line and dashed line in Fig.7.

4.2 Distribution of strain in reinforcement

Fig.12 shows a distribution of the curves of the load, P, versus the reinforcement strains at designated locations for SL-72 from the experiment and the analysis. The following observations can be drawn from these figures: (1) the top reinforcements experienced almost no tensile strains, with the exception of a small amount of tensile strain at the ultimate-load stage, though the bars were designed to yield in tension. (2) with regard to the bottom tensile reinforcements, the gauges, SG11 and SG12, showed a negligible increase in the tensile strain, while SG13 and SG14 showed clear yielding at $P_y (=470kN)$ through large tensile strains that exceed the yield strain at the location under loading point P2. This implies the alleviation of anchorage requirement in support A. (3) the tensile strains at the vertical shear reinforcements (at the middle of the depth) were not significant until the yield point. This indicates that shear forces are transmitted by concrete strut and the failure of the specimens was governed by the crushing of the concrete strut.

4.3 Crack development and failure modes

In the case of AL-86, flexural cracks occurred around the mid-span at a load of 274kN, and a diagonal tensile crack occurred in the shear-critical region at a load of
480kN, with a small drop and recovery of the load. The number of flexural cracks then increased as the load reached 650kN. When the load exceeded 650kN and continued to increase, the number and range of diagonal cracks in the shear-critical regions increased. However, under a load of 778kN, the number of cracks stabilized when the diagonal cracks extended to flat cracks in the area under the upper wall. Above this load, only the width of one major vertical crack under the load point, P2, continued to increase up to about 6mm, and the specimen finally failed due to the crushing of concrete at the top end of the diagonal strut.

Specimen SL-72 showed an initial flexural crack at a load of about 290kN. The number and range of flexural cracks increased up to a load of 457kN when the first diagonal shear crack occurred in the shear-critical region. The range and number of diagonal shear cracks increased as the load increased from 505kN to 637kN. At a load of 643kN, the ends of the diagonal cracks at the upper left-hand portion of the shear-critical region extended to become somewhat flat beneath the wall corner and led to local failure due to the compressive crushing. After this local failure, the load decreased rapidly to 330kN with a large widening of the major shear diagonal cracks, which finally led to the fracture of the vertical shear reinforcement. Figs.13.(a) and (b) show the crack patterns of AL-86 and SL-72, respectively. The deformation of the supporting steel plate in AL-86 due to the high compressive force transmitted from the strut can be found in Fig.13.(c).

5. Analytical Simulation of the Test Results
5.1 Analytical modeling

The objective of this analytical simulation of test results is twofold: (1) to obtain information that is not provided by the test results, such as the effects of support rigidities and internal force distributions; and (2) to verify the reliability of the currently available nonlinear-analysis software in the prediction of the nonlinear behavior of concrete structures. For these objectives, a nonlinear finite element analysis program, DIANA\textsuperscript{15}, was used. The analysis results will be presented mainly with regard to specimen SL-72. However, whenever necessary, the case of AL-86 will also be presented. The material model of Drucker-Prager yield criteria with tension cut-off, for which the values of parameters are given in Table 4., was used for concrete. Line elements are assigned to the reinforcements with Von-Mises yield criteria. Four-node quadrilateral isoparametric plane stress elements are mainly used for the FE meshes. The diagonal from the upper-left corner of the shear span to the lower-right corner that is adjacent to the steel plate of support B appeared to be a major location of diagonal cracks. Therefore, interface elements were inserted into this diagonal. Thus, three-node triangular isoparametric plane stress elements were selected for the shear span.
Quasi-Newton (secant) method for solving nonlinear equations and Internal energy criterion for the convergence criterion were chosen.

Regardless of how accurately an experimental set-up is implemented, a level of flexibility or gap between the supports that restrain the specimen and the specimen itself is in reality inevitable. In particular, if the restraining support faces downward at the top, and is therefore passive at the right end of specimens, as it is in this experiment, the initial gap may be inevitable with the application of downward loads. This is a disadvantage of this loading set-up. It is therefore assumed that all the supports, A, B, and C have springs with step functions of spring constants. Fig.14. shows a comparison of the final \( P-\Delta_m \), \( R_y-\Delta_m \), and \( P-\Delta_c \) curves with those from the experiment for both the AL-86 and the SL-72 specimens after adjusting arbitrarily the step functions of spring constants to best fit the experimental results. And, Fig.12. compares the analytic results with the experimental strains for SL-72. From the comparison in Fig.12. and Fig.14., the correlations between experiment and analysis are satisfactory.

5.2 Distributions of deformation, strain, cracks, and stress

The distributions of the deformation, cracks, and principal strains in the shear-critical region of the state of imminent collapse for SL-72, which are obtained through analysis, are shown in Fig.15. The distributions of the deformation and cracks are similar to the test results for SL-72 shown in Fig.13.(b). The distribution of the principal strains presents very large horizontal tensile strains \((\varepsilon=406\times10^{-6}\text{m/m})\) along the bottom horizontal bars. Very large principal tensile and compressive strains \((\varepsilon=278\times10^{-6}\text{m/m})\) also occur in the left upper corner of the shear zone in Fig.15. (c). The failure mode that was caused by concrete crushing in the experiment could not be simulated due to the limitation of the Drucker-Prager model used. The distributions of the Von Mises stress at the states of imminent collapse are given for both the AL-86 and the SL-72 specimens in Fig.16., where it can be seen that the center of rotation is located immediately beneath the load point, P2, and that the length of \(jd\) and the angle, \(\theta\), in the behavior model of phase I as shown in Fig.10.(a), are therefore much larger than those assumed in Table 3. The underestimation of the arm length of the bottom longitudinal tie with respect to the center of rotation and the angle of the diagonal concrete strut appears to be the main reason for the large difference in the yield bending moment and the yield shear force between the experimental results and the strut-and-tie estimations in Table 3.

6. Conclusions

The design of deep girders that transmit the gravity load from the upper shear wall to the lower columns has always been a challenging task for practicing structural engineers since it is known that while beam theory cannot be applied to this case, the strut-and-tie design approach has not been fully elaborated and therefore cannot be used with confidence.

Based on the experimental and analytical study, the following conclusions are drawn.

1) The conventional design approach that is based on Bernoulli’s principle does not provide adequate information on the real behavior of reinforced concrete deep girders and their connections. The approach of the strut-and-tie method is valid for this type of girder.

2) Since the upper load was carried over directly to the supporting columns through the stiff concrete strut without any assistance from the adjacent beam up to the point of yielding of the bottom ties, the shear capacity of continuous deep girders is mainly governed by the bottom tie yielding forces, and up to the yielding of the bottom tie, it is reasonable to model the continuous deep beam as a group of independent simple deep beams.

3) The additional shear resistance derives from continuity with the adjacent beams or walls. Shear and top longitudinal reinforcements in the continuity region with adjacent girders can be designed by using appropriate available models, including the strut-and-tie model, for the additional margin of safety that incorporates strength and ductility.
Simulation through nonlinear two-dimensional analyses using DIANA shows good correlations with the experimental results. The effect of the support rigidity and boundary conditions on the behavior of test specimens could be quantified. The stress distributions at the state of imminent collapse reveal that the arm length between the center of rotation and the bottom tie can be significantly larger than that assumed in the strut-and-tie model. Therefore, a more reliable model of struts and ties needs to be developed. Finally, it is believed that the effects of continuity and support conditions that differ from those of the tests reported herein on the behavior of the general continuous reinforced concrete deep girders can be studied by using this analytical approach in the future if the crushing failure of concrete could be simulated.

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