A CASE STUDY ON STRENGTH EVALUATION OF STEEL CONNECTION ASSEMBLIES EMBEDDED IN PRECAST MEMBERS

James A. Attenhofer, Clemson University, Clemson, SC
Tommy Mitchell, PE, Tindall Corporation, Spartanburg, SC
Brandon E. Ross, PhD, PE, Clemson University, Clemson, SC

ABSTRACT

This paper is a case study in the application of ACI 318-05 Chapter 20, Strength Evaluation of Existing Structures, to evaluate steel connection assemblies embedded in precast members. The evaluation described in the paper was conducted to address a purchaser's concerns over the level of quality assurance used for welds in the steel assemblies. These concerns were expressed after the members were already fabricated but before they were completely erected. Because the welds in question were embedded in concrete they could not be visually observed, nor could they be directly tested without destroying the precast members. In lieu of direct testing of the welds, a load test regime was conducted based on ACI chapter 20. Load tests were informed by structural analyses, which were also a primary feature of the evaluation. Details and results of the structural analyses, load tests, and application of ACI chapter 20 are discussed.

Keywords: Strength Evaluation, Concrete Cracks, Load Testing, Welds, Steel Connection Assemblies, ACI 318-05 Chapter 20

INTRODUCTION

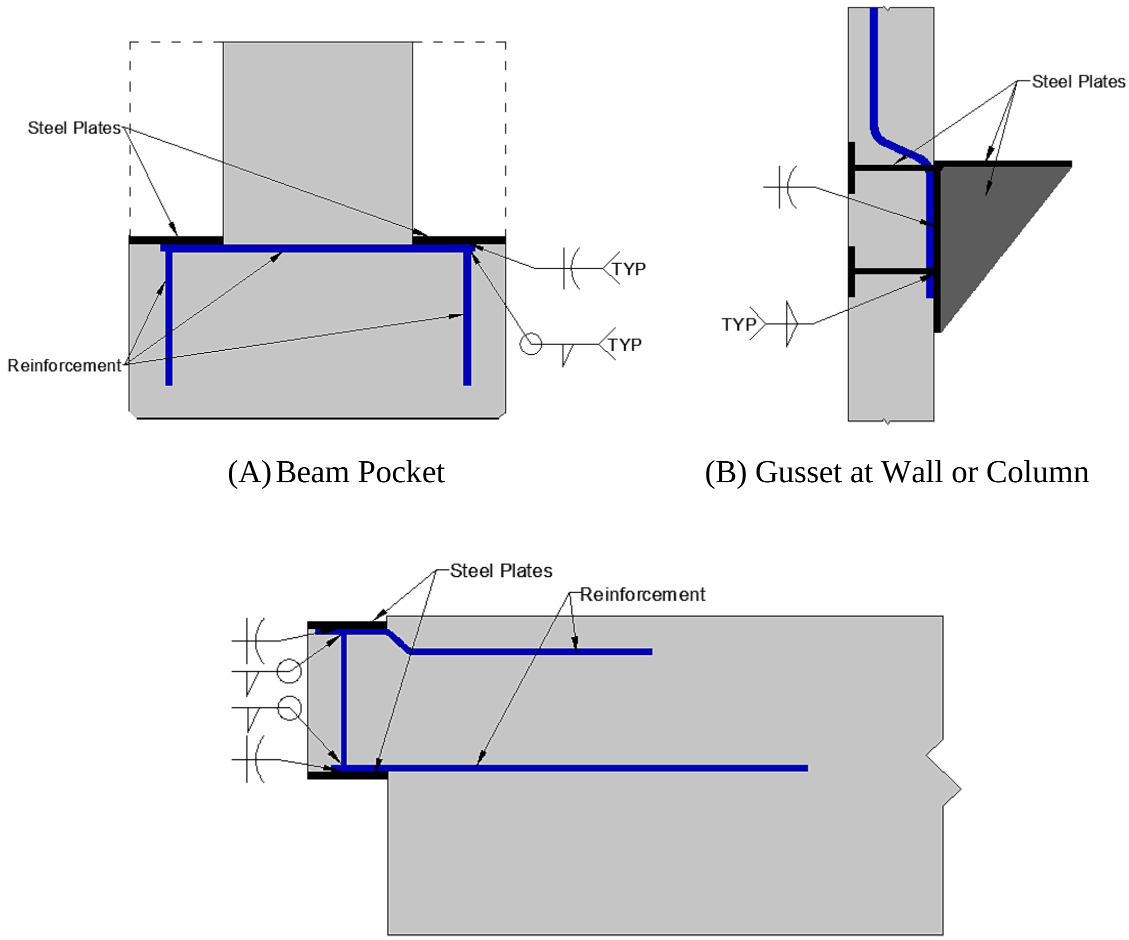
The following paper is a case study on the evaluation of weld strength in steel connection assemblies embedded in precast concrete members. The program was initiated to address a purchaser's concerns over the level of quality assurance of the welds in the connection assemblies. The assemblies in question were for connections in precast concrete members. Concerns regarding the quality assurance were brought to the attention of the precast supplier after the members were already fabricated, but before they were erected. This paper does not consider the validity of the purchaser's concerns, but rather focuses on the actions taken to alleviate those concerns. Specifically, experimental and analytical evaluation programs were conducted, with chapter 20 *Strength Evaluation of Existing Structures* of ACI 318-05[1], serving as the basis for evaluation.

This case study describes the experimental and analytical methodologies used in the program, which were developed in accordance with the commentary from ACI R20.1:

> *If the safety concerns are related to an assembly of elements or an entire structure, it is not feasible to load test every element and section to the maximum. In such cases, it is appropriate that an investigation plan be developed to address the specific safety concerns. If a load test is described as part of the strength evaluation process, it is desirable for all parties involved to come to an agreement about the region to be loaded, the magnitude of the load, the load test procedure, and acceptance criteria before any load tests are conducted.*

Four primary parties were involved in the evaluation program. The first party was the purchaser, who will not be mentioned by name. Two structural engineers acting as the purchaser's representatives were assigned to evaluate and observe all phases of the program. The second party was the primary supplier of the overall project, who will not be mentioned by name. Representatives of the primary supplier observed work performed during testing. The third party was the secondary supplier, Tindall Corporation, who served as the precast fabricator and erector. Tindall Corporation's Chief Engineer served as the engineer of record for the precast system. Tindall Corporation will be referred to as the "fabricator", and Tindall Corporation's Chief Engineer will be referred to as the "structural engineer of record" for the remainder of this paper. "Structural engineer of record" (SER) will be used when discussing engineering responsibilities and tasks, and "fabricator" will be used in all other instances. The final party was a representative from the Glenn Department of Civil Engineering at Clemson University, who was selected by the purchaser's representative and SER to act as an independent consultant. The representative from the Glenn Department of Civil Engineering will be referred to as the "consultant" in this paper. Collectively the purchaser's representatives, SER, fabricator, and consultant will be referred to as the "evaluation team."

The case study involves a three-story, 64,700 sf plan area, industrial facility built almost entirely of precast beams, columns, wall panels, roof panels, and frames. In total, 1,433 precast members were used in the structure. Connections between precast members were facilitated using embedded steel connection assemblies commonly used in the industry. Individual assemblies were comprised of multiple plates and reinforcing bars connected by welds (Figure 1). The evaluation program focused on the strength of these welds. Connections typically fit in one of the general categories shown in Figure 1. The structural layout facility was uniform and typical member sizes and details were used throughout. The design basis for the facility was the 2006 IBC[2] and ACI 318-05. Connections were designed using the 6th edition of the PCI Design Handbook[3].



(A) Beam Pocket (B) Gusset at Wall or Column

(C) Dapped Beam

Figure 1 - Connection assemblies

Steel connection assemblies were built in-house by the fabricator. Quality Control provided for the welds in the assemblies was consistent with PCI MNL-116 Division 6 provisions[4]. This level of inspection and documentation is typical for the precast industry. Although PCI MNL-116 provisions were originally approved by the purchaser for use on the project, the purchaser subsequently asserted that they anticipated a more stringent degree of inspection and documentation based on their interpretation of AWS D1.6 provisions[5]. To address the purchaser's concerns, the purchaser's representative and SER mutually agreed to conduct an experimental and analytical program to evaluate the welds in question. The independent consultant was retained to assist in the design and execution of the evaluation program.

PRELIMINARY ANALYTICAL PROGRAM

The first stage in the evaluation program was a review of all design calculations performed by the SER. The review was conducted by the independent consultant and included the evaluation of loads and capacities for each connection type. In the few minor instances where discrepancies were observed in the calculations, they were resolved through discussions between SER and consultant. These discrepancies resulted in differences of capacity less than 2%, and differences in load less than 16%. The loads calculated by the SER were typically larger than those calculated by the consultant, and were the result of simplifying, but conservative, procedures used to determine tributary areas. The conservative values from the SER were used for subsequent analyses.

After loads and capacities were verified, a demand-to-capacity ratio (DCR) was calculated for the welds in each type of connection using Equation 1. DCR provided a quantified means of assessing the criticality of deficient welds in each connection. A DCR of 1.0 meant that the nominal weld strength was equal to the factored load; a DCR of 0.5 meant that the nominal strength was twice the factored load. Conversely, a DCR of 0.5 also meant that 50% of the weld could be defective or omitted and the nominal weld capacity would still be equal to the factored load. A strength reduction factor of 0.75 was used in the weld design calculations, meaning that a DCR of 0.75 or smaller was needed to satisfy minimum code requirements. Values for DCR ranged from 0.11 to 0.46.

$$DCR = \frac{R_u}{R_n} \qquad \text{Equation 1}$$

Where:

R_u is the maximum factored load supported by the welds in the connection assembly

R_n is the nominal capacity of welds in the connection assembly as specified by SER

Tested strength of electrodes was confirmed by reviewing documentation from the material supplier. The specified weld electrode material was 100 ksi. Documentation by the material supplier reported typical electrode strength of 109ksi. The conservative specified value was used when calculating DCR.

The same types of connection assembly were used in multiple places throughout the structure. In these instances, the factored load from the worst case was used to calculate DCR for a given connection type.

Welds in the connection assemblies were designed to have greater capacity than the components being connected. The components, however, were not the subject of concern for the purchaser. Thus the nominal strength of the welds -not the components- was used to calculate DCR. This approach was taken so that the DCR would highlight conditions and connections where deficient welds would be of greatest concern.

EXPERIMENTAL PROGRAM

After completion of the preliminary analytical program, the evaluation team held a meeting to review the preliminary analysis and to determine a direction for the experimental portion of the program. During the meeting, all parties agreed upon the test procedures and criteria described in the following sections.

BASIS, SAMPLING, AND LOADING

ACI 318-05 Chapter 20, *Strength Evaluation of Existing Structures*, was used as a basis for the experimental program. The ACI committee 437 report, *Load Tests of Concrete Structures: Methods, Magnitude, Protocols, and Acceptance Criteria*, was also consulted to design the experimental program[6]. Evaluation was limited to those connections with a DCR equal to or greater than 0.25. At this threshold, 67% of the weld could be deficient or omitted and the nominal strength multiplied by the strength reduction factor would still be greater than the factored load. By only testing connection assemblies with DCR greater than 0.25, it was implicitly assumed that the fabricator consistently provided at least 67% of the specified weld.

Due to the difficulty of removing embedded assemblies for direct evaluation and testing, assemblies were indirectly evaluated by testing the precast members holding the assemblies. This approach can be described by making analogy of the precast members to a chain. In this analogy, each link in the chain represented a component of the load path through a precast member. Chain links included the bearing plate, weld, reinforcement bars, and concrete. It was assumed that load testing would manifest problems in the weakest link. If the links were sufficient to support the test loads, then it was concluded that the connection system, including the welds in question, had adequate capacity. This approach limited construction delays because it did not require removal of assemblies from previously fabricated members. Members that were not damaged during load testing were permitted to be used in the structure.

The number of connections that were evaluated was based on ACI 20.2.2, which sets the requirements for identifying sizes and spacing of reinforcement in existing structures. The commentary for this section states that in large structures, determination of reinforcement details at 5% of the critical locations "may suffice if these measurements confirm the data that was provided in the construction drawings." Based on this commentary, all parties in the evaluation team agreed that adequacy of the connection assemblies would be confirmed by testing 5% of critical locations. This interpretation may not be applicable in other circumstances and should be evaluated on a case-by-case basis. Because the same types of connections assemblies were used throughout the structure, “critical locations” were defined as those locations for each connection type that had the largest design loads.

The test loads were calculated using the design loads from the critical locations and the load combinations from ACI 20.3.2. Based on ACI 20.4, loads were applied to the members in four approximately equal stages and the maximum load was held in place for 24 hours.

ACCEPTANCE CRITERIA

The accept/reject criteria of 20.5.2 are based on deflections. To make the criteria applicable to testing of connection assemblies, the requirements were modified to consider crack width in lieu of deflection. This modification is consistent with the commentary from R20.5.2 which acknowledges that “In the case of a very stiff structure, however, the errors in measurements under field conditions may be of the same order as the actual deflections and recovery.” Furthermore, section 20.4.1 includes crack width as one of the response measurements to be considered in testing.

One deflection criterion from 20.5.2 is that the structure must recover at least 75% of peak deflection after the load is removed. The evaluation team interpreted this to mean that an acceptable connection assembly should exhibit significant elastic recovery after being subjected to the prescribed load. Accordingly, a crack width criterion was established that required connections to exhibit elastic recovery. Following the form of ACI equation 20-2, Equation 2 was established for determining acceptable residual crack width:

$$W_r \leq \frac{W_1}{3} \qquad \text{Equation 2}$$

Where:

W_r is the residual crack width after load has been removed

W_1 is the maximum crack width under ACI chapter 20 applied load

Equation 2 requires that 67% of the peak crack width be recovered upon removal of the load. Failure to close the crack to at least 67% suggests that some portion of the connection experienced unacceptable plastic deformation during loading, and that the connection was near its ultimate capacity. The 67% recovery requirement for crack width was less stringent than ACI equation 20-2 requires for deflections. The reason for the reduced requirement was to account for the possibility of concrete debris lodging in a crack and restraining closure. The evaluation team chose 67% recovery of crack width as a compromise between elastic recovery and the possible effects of debris. This decision was of minor consequence in the test program because almost all of the tested connections had either no cracking or had greater than 75% recovery.

The qualitative acceptance criteria of 20.5 were also applied to the test program. These criteria included compression failure (20.5.1), shear failure (20.5.3), inclined cracking (20.5.4), and bond failure (20.5.5). A maximum crack with of 0.04 in. was also imposed as an acceptance criterion. This value was the crack width threshold for serviceability of the structure as set by the SER. The project specifications required repair of cracks greater than 0.008 in. in width, and the SER selected half of that value for the serviceability limit. By imposing this limit, the project team enforced serviceability requirements as well as strength requirements. Qualitative acceptance and rejection criteria are summarized in table 1.

Table 1 – Qualitative Accept/Reject Criteria

Acceptable test	Rejectable test
Cracking does not occur.	Extensive cracking occurs.
If cracking does occur, the maximum crack widths are less than 0.04 in. and cracks larger than the serviceability limit close after the load is removed. (ACI R.20.5.1, 20.5.2)	Crack width exceeds 0.04 in. and/or cracks do not close significantly after the load is removed. (ACI R.20.5.1, 20.5.2)
The assembly does not collapse and is able to support the applied load throughout the test without concrete spalling or crushing. Minor surface scaling around embed is acceptable. (ACI 20.5.1)	The assembly collapses or is otherwise unable to support the applied load throughout the test. Or, concrete spalls or crushes during testing. (ACI 20.5.1)
Cracking in anchorage and/or lap splice regions shall not indicate imminent failure. (ACI 20.5.5)	Cracking in anchorage and/or lap splice regions indicate imminent failure. (ACI 20.5.5)
Plastic (permanent) displacements of steel elements indicate ductile behavior of the connections and shall not automatically result in rejection. Test members having excessive ductile displacement shall not be installed in the structure. (ACI R20.5.1)	

TESTING

Load test procedures of ACI 20.3 and 20.4 were used in the program. Loads were applied in four approximately equal stages and response measurements were taken after each stage. Testing took place in the fabricator’s storage yard. Precast members that were not being evaluated were used to apply the load (Figure 2). The total load was held in place for 24 hours, following which additional response measurements were taken. Loads were calculated as from the combinations given in ACI 318-05 section 20.3.2.

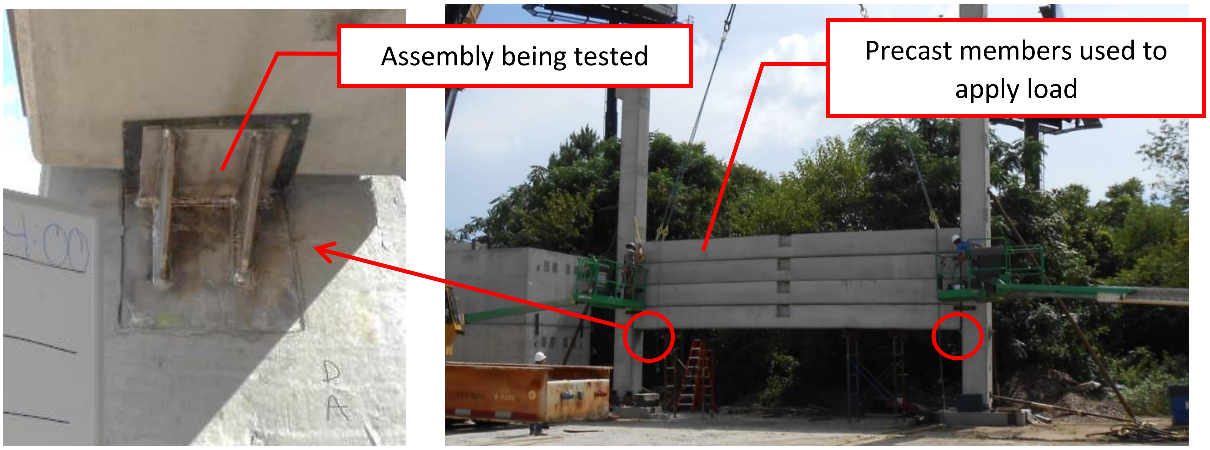


Figure 2 - Application of load to gusset assembly

Multiple connections were tested in a single test set-up.

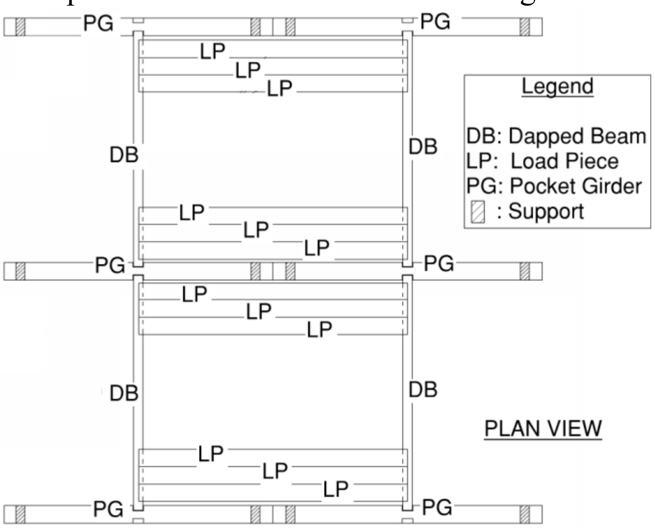


Figure 3 shows the setup for testing assembly connections at dapped ends beams and pocket girders. In this test set-up, 8 dapped beam connections and 8 beam pocket connections were tested. The wall and column test setups tested only two connections each load test, one on each wall or column (Figure 2). The tests were conducted by the fabricator with the same crew used for erection of the industrial facility. The set-up tolerances used in the tests were consistent with those used during erection of the facility (PCI MNL 135-00)[7]. Testing was observed by the SER, purchaser's representative, and the consultant.

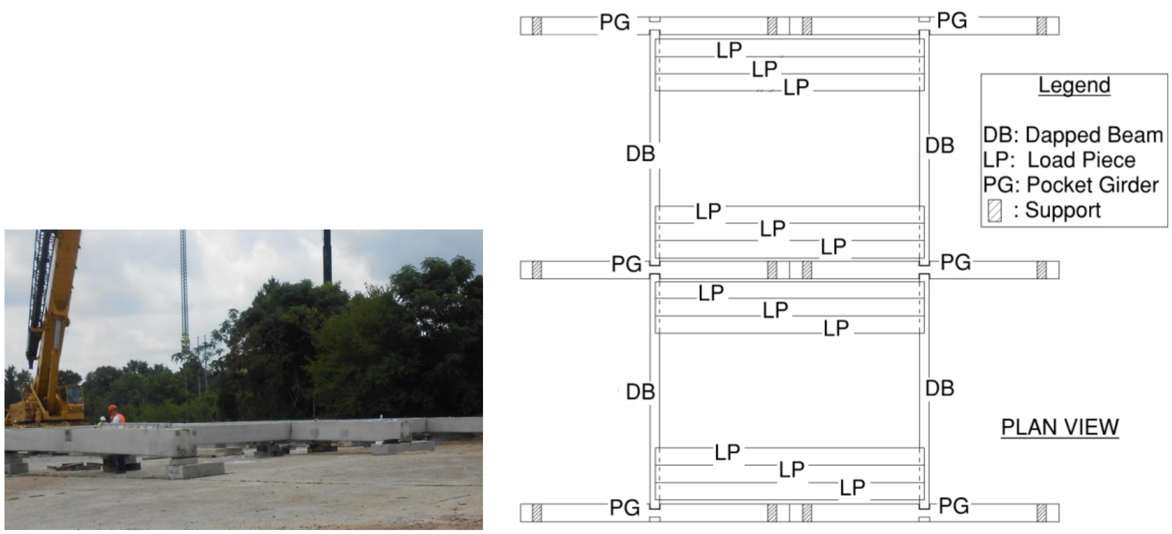


Figure 3 – Dapped beam and pocket girder test setup

Load pieces used were placed on the connections using a crane. The weights of the pieces were calculated using known member dimensions and concrete unit weight, and verified using a load cell on the crane. As a safety measure, steel shoring was placed below the load pieces during the loading process. Shoring was placed with a small gap below the load pieces so as to provide support in the event of a failure, but to not attract load during testing (Figure 4).

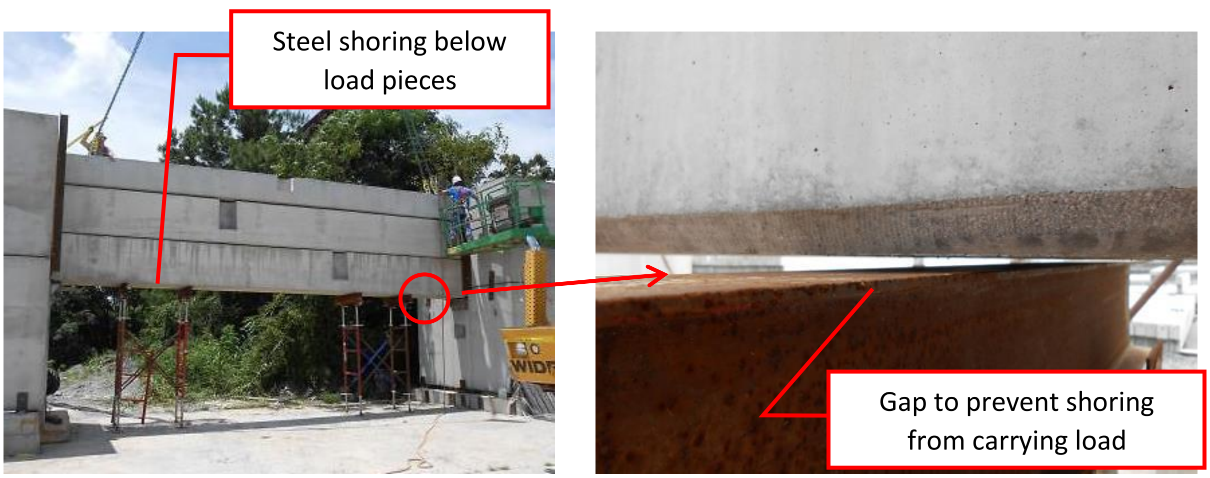


Figure 4 - Shoring below precast member during testing

Each connection was evaluated for cracking and/or other indications of structural damage at the following milestones:

- Immediately before testing
- After each of the four loading stages
- 24-hours after full load was placed

- Immediately after all loads were removed

When cracks were observed, they were measured, marked, and photographically documented (Figure 5). Cracks widths were measured using a microscope that was precise to +/- 0.001 in. When the microscope could not be used due to physical constraints (e.g. microscope could not physically fit over the crack due to a conflict with assembly or member), a crack comparator card was used. Figure 6 (right) shows the use of a comparator card to measure a crack that was too close to the gusset plate to be readable with the microscope. A mark was placed on the concrete face to ensure that crack width measurements were also taken at the same location. To further ensure consistency in crack measurements, the same person always measured crack widths on a given precast member.

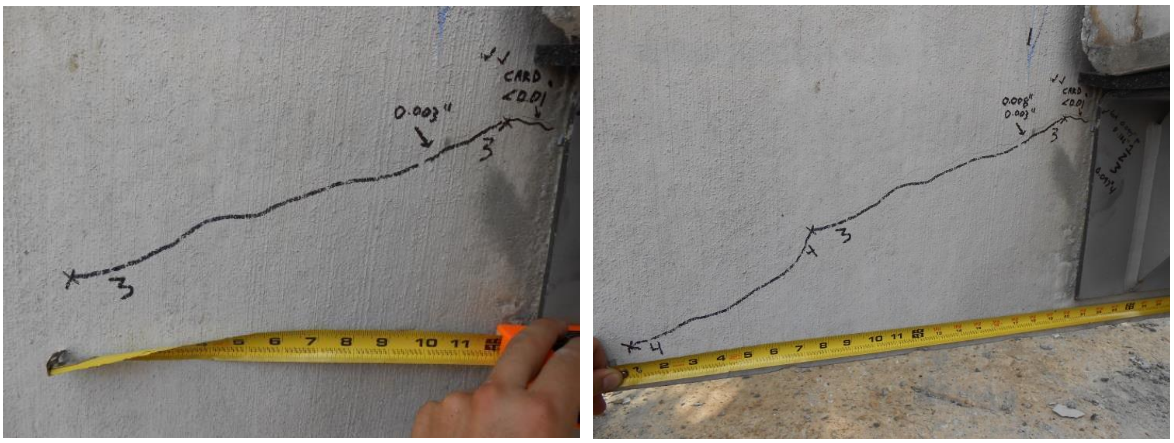


Figure 5 - Documentation of cracks after load stage 3 (left) and load stage 4 (right)

Figure 6 - Measuring crack with microscope (left) and card (right)

Attempts were made to evaluate the behavior of gusset plate assemblies (Figure 1B) by measuring the gap between the concrete surface and back of the connection plate. The approach was to use steel plates of known thickness to measure the increase/decrease in the gap as the load was applied/removed (Figure 7). This approach was unsuccessful because the steel plates were not precise enough to practically measure changes in gap size under field conditions. Furthermore, in some members the gap could not be measured because the back of the gusset was embedded in concrete. In the end, the evaluation team decided to abandon gap measurements as a means of evaluation.

Each of the connection assemblies tested in the program satisfied the acceptance criteria established by the evaluation team. Had any of the connections failed, then a reloading test would have been conducted within 72 hours of the first test, as per ACI 20.5.2.

Figure 7 - Measuring gap behind gusset plate

REPORTING

A report was prepared for each test setup, and included the following information:

- Identification for each member and connection assembly in the setup
- Date of loading and unloading
- Name of each observer
- Weight of each load piece and time of placement/removal
- Overall picture of the setup during each load phase
- Pictures of each connection during each load phase
- Pictures and descriptions of cracking and/or other damage(if applicable)
- Crack width measurements at each connection during each load phase (if applicable)
- Calculations of experimental and allowable experimental crack width (if applicable)
- Statement regarding pass or fail of each connection assembly

These reports were prepared by the consultant and submitted to the purchaser's representative for review and approval. To expedite acceptance of the reports, the purchaser's representative provided unofficial reviews during report preparation.

RELIABILITY ANALYSIS

The consultant has recently completed a reliability analysis of the entire structure to augment results from the experimental program. The overall goal of the reliability analysis was to use probabilistic methods to determine the likelihood that a deficient weld would result in a structural failure. A detailed discussion of the methodology and results will be described in a forthcoming publication. The reliability analysis supported the conclusion from the experimental program that the tested connection assemblies are acceptable for use in the structure.

SUMMARY AND LESSONS LEARNED

A case study has been presented on strength evaluation of steel connection assemblies in precast concrete members. The evaluation was conducted to address a purchaser's concerns over the level of quality assurance for welding in the connection assemblies. The welds in question were embedded in concrete and could not be directly observed or tested without destroying the housing member. Experimental and analytical methods, based in part on ACI 318-05 Chapter 20, were used to indirectly evaluate the welds and resolve the purchaser's concern. The following lessons learned may be useful for other parties undertaking similar evaluation programs:

- **Collective effort from the evaluation team**. The purchaser's representative, structural engineer of record, fabricator, and consultant worked together to achieve resolution of the purchaser's concern. Sampling protocols, test methods, and accept/reject criteria were collectively established and rigorously defined prior to embarking on the evaluation program. Differences of opinion - which certainly did occur - were resolved through continuous and respectful communication.
- **Application of ACI 318-05 chapter 20**. The provisions of ACI 318-05 chapter 20 provided a baseline for conducting and analyzing load tests. The commentary associated with this chapter was particularly valuable in determining how to apply the code provisions to the conditions being evaluated.
- **Conservative weld design**. Welds specified by the engineer of record had nominal strengths that were two to ten times greater than the factored loads carried by the welds. Primarily, the reasons for the excess weld design strengths were assumptions of lower electrode strengths in the original design and intentional over sizing of the weld to assure failure modes by ductile steel elements. Although the conservative designs do not imply anything regarding the quality of the welds produced by the fabricator, the conservative designs gave a greater margin of error in the event that the welds were deficient.
- **Focus on critical conditions**. The demand-to-capacity ratios calculated in the preliminary analytical program were useful in identifying connections that were most likely to fail in the event of a deficient weld. This information was used to target the most critical connections for the subsequent test program.

REFERENCES

1. ACI 318-05 Building Code Requirements for Structural Concrete and Commentary. American Concrete Institute. Farmington Hills, MI. 2005.
2. IBC International Building Code. International Code Council. 2006.
3. PCI MNL 120 Design Handbook 6th Edition. Precast/Prestressed Concrete Institute. Chicago, IL. 2004.
4. PCI MNL 116 Manual for Quality Control for Plants and Production of Structural Precast Concrete Products, 4th Edition Precast/Prestressed Concrete Institute. Chicago, IL. 199.
5. AWS D1.6 Structural Welding Code Stainless Steel. Miami, FL. 2007.
6. ACI 437.1R-07 Load Tests of Concrete Structures: Methods, Magnitude, Protocols, and Acceptance Criteria report. American Concrete Institute. Farmington Hills, MI. 2007.
7. PCI MNL 135-00 Tolerances Manual for Precast and Pre Stressed Concrete Construction, Precast/Prestressed Concrete Institute. Chicago, IL 2000.