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Assessing and evaluation of the mechanical properties of reinforcement coupler systems in Turkey

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ABSTRACT: As the size of structures become super larger in the current construction trends, construction engineers have tried to enhance the quality of reinforced concrete structures, the simplification of field works and reduction of construction period. Connection of reinforcing rebar for concrete structures is also an important issue for super-sized structures. Most common rebar connection methods available are the lap splice joint, pressure welded joint, and the mechanical joint methods. When thick rebar's are required to be connected, however, design codes such as ACI-318 recommends the lap splice joint to be avoided. This is because of potential difficulties in placing concrete in complex rebar area and securing sufficient cover depth, and uncertainty of bond between rebar and concrete. The welded pressure joint method has some inherent shortcomings in quality control by heating rebar ends and pressure welding them together.

This study will present the results of independent study of mechanical reinforcement splices conducted at the Anadolu University of splicing systems marketed in Turkey. The results of this testing program have shown that mechanical splices are indeed capable of effectively connecting reinforcing bars together and that many are capable of producing bar breakage as the failure mode; thus indicating that the splice is stronger than the parent bar itself.

1 INTRODUCTION

Use of mechanical connection systems in reinforced concrete has become increasingly prevalent in Turkey. Mechanical connectors are an alternative to lap and welded splices, and many arecapable of developing the full strength of the connected reinforcing bars. There are many advantages for using mechanical connector systems over conventional reinforcing bar lapping. Such examples are overcoming reinforcement congestion problems and convenience when installing precast construction members at sites.

The purpose of this research is to establish the unsuitability of the current Turkish reinforced concrete design standard's approach to the performance verification of mechanical connectors.

This paper presents:

- Findings on international trends of mechanical connection systems;
- Recommendations for Turkish reinforced concrete standard review; and
- Indicative test results using newly proposed testing protocol.

2 REVIEW OF DESIGN PROVISIONS FOR MECHANICAL SPLICES

International literature reporting on mechanical connection testing protocols and experimental studies conducted in the United States, Japan, and Europe were assessed to provide recommendations for Turkish reinforced concrete standard. Both static and seismic conditions, in terms of their relevance in the Turkish context, were examined in this literature review.

2.1 Static conditions

Based on assessment of the international literature, a general conclusion was drawn. For use of mechanical connection systems in static conditions, three categories need to be considered, and they are:

- Strength;
- Serviceability limit state; and
- Fatigue loading.

2.1.1 Strength:

The general trend for strength requirement of mechanical connectors in static conditions is that the strength of the connectors must be larger than that of the spliced reinforcing bars. Most reinforced concrete design standard organizations demand an over strength factor to be multiplied to the specified yield strength of the spliced reinforcing bars for the connector's strength requirement. This over strength factor tends to be governed by both safety based on reinforcing bar manufacturer's quality control and on economy considerations. Some organizations are more restrictive than others in this matter, as they require the connector strength to be more than the specified ultimate strength of the spliced reinforcing bars. The basic logic behind the above requirements is that the spliced reinforcing bars must yield and eventually fail before the ultimate failure of mechanical connectors under loading situation, thus avoiding brittle failure of the connectors.

2.1.2 Serviceability Limit State

A number of reinforced concrete design standards recognize possible concrete cracking, which may arise from slip between the spliced reinforcing bar and the mechanical connector, thus constituting a serviceability limit state. It is understood that this slip, which is a permanent or residual deformation, is a matter of manufacturer's quality control on interlock between the spliced reinforcing bar and the connector. It also is understood that the mechanical connection system tends to be more rigid once it is subjected to low stress. The reason for this is the plastic deformation due to bearing within the interlocked system of the reinforcing bar and the connection. Serviceability limit state design in reinforced concrete under static conditions is an important aspect that structural engineers need to consider. Appropriate crack width limits provide an aesthetically sound environment for the public as well as preventing possible corrosion occurrence in reinforcing bars.

2.1.3 Fatigue Loading Situation

A few organizations require fatigue testing of mechanical connection systems. This fatigue loading, which is a high number of cycles within the elastic stress range, can affect the mechanical connector's performance. However, assessing fatigue behavior of the mechanical connectors was beyond the scope of the research reported here.

2.2 Seismic conditions

The design provisions for mechanical splices in Turkish code is not enough to design a connection using mechanical coupler in applications.

In British Standard BS 8110: Part 1, the tensile strength of coupled hot-rolled bars has had to be at least 1.15 times the nominal yield stress, f_{sy}. The intention has been that the coupler should be stronger than the bar [12]. In practice the requirement in BS 8110: Part 1 has only meant that the strength of a splice at least equals the required minimum tensile strength of the bars. As a consequence, it has been conceded that it may not be possible to force failure outside a coupler when the bar strength exceeds the minimum. Another requirement in BS 8110: Part 1 has been to ensure that the permanent elongation across the coupler does not exceed 0.1 mm after the bars have been stressed to $0.6f_{sy}$. This elongation arises due to slip of the bars relative to the coupler. It appears likely that the value of 0.1 mm was chosen because it was considered to be sufficiently smaller than the normal upper characteristic limit of 0.3 mm for crack widths in buildings, and does not appear to be supported by the results of research .

In NZS 3101: Part 1, mechanical splices are normally required to be stronger than the breaking strength of the bars. This may require the coupler housing to be very strong indeed, and for tests to be done using very high-tensile bars to demonstrate conformance, AASHTO (1996). Unfortunately, provisions explaining to manufacturers how they must test their products for conformance are not contained in NZS 3101: Part 1. The New Zealand Standard also allows mechanical splices with less capacity to be used in "partial splices" provided a number of conditions are met, i.e. they are staggered, can develop at least twice the calculated force in the bars (i.e. bars have excess capacity), etc.

In the United States today there are several different codes that these splice manufacturers must meet or at least consider. These codes include the ACI 318 [1995, 1999], the Uniform Building Code [ICBO 1997], the BOCA (Building Officials and Code Administrators) Building Code [1999] and the Southern Building Code [SBCC 1997]. Moreover there are a separate set of code for bridges, principally the AASHTO (American Association of State Highway Transportation Officials) Bridge Design Specification [AASHTO 1998], as well as various state standards for the design of bridges. Lastly there arefederal standards written by the U.S. Army Corps of Engineers and other agencies.

Given the complexity of the code landscape in the US, the splice requirements are surprisingly similar. All of the various code bodies in the US follow, to some extent, the basic reinforced concrete code, the ACI 318. Here the traditional requirement for both welded and mechanical splices has been that the splice be capable of developing 125% of the nominal yield strength of the spliced bars, Figure 1. This requirement is intended to ensure that the splice will be able to develop the yield strength of the bar. The uncertainty associated with the actual yield strength of the reinforcing bars is large and thus there must be an increase in the required strength of these welded and mechanical splices; especially since there are no other governing standards involved. The actual yield point in reinforcing bars can exhibit yield strengths of 25% or more above nominal yield values.

As a special note, in the new 1999 edition of the ACI 318 Building Code, the requirement for mechanical and welded splices in nonseismic applications has stayed at 125% of nominal yield. However, in structures with increased seismic performance requirements, mechanical and welded splices are required in the ACI 318-99 to develop the nominal tensile strength of the bar [ACI 1999]. No cyclic tests are required but the splice must exhibit this increased level of strength, and by implication an increased ductility as well.

12.14.3 — Mechanical and welded splices

12.14.3.1 — Mechanical and welded splices shall be permitted.

12.14.3.2 — A full mechanical splice shall develop in tension or compression, as required, at least **$1.25f_y$** of the bar.

R12.14.3 — Mechanical and welded splices

R12.14.3.2 — The maximum reinforcement stress used in design under the Code is the specified yield strength. To ensure sufficient strength in splices so that yielding can be achieved in a member and thus brittle failure avoided, the 25 percent increase above the specified yield strength was selected as both an adequate minimum for safety and a practicable maximum for economy.

Figure 1. ACI318 mechanical splice performance requirements.

3 TEST METHODOLOGY

A discussion of the proper method to reasonably test mechanical splices ensued. So as to provide a simple procedure, an in-air test was selected. While this is not as accurate as an in-member test, it would provide a conservative lower bound for the performance. It was decided to limit the bar size to 26-mm, 32 mm and 40-mm in diameter and to use only Grade 420 steel with a nominal yield of 420 MPa.

The so-called ACI 439 Test Procedure consisted of three separate tests intended to determine how well the spliced bar system performed under simulated seismic loading. A test specimen with a gage length of r approximately 800 mm for these bars, was used to simulate the length of a plastic hinge that would be found in a flexural member. The goal was to determine the response of the specimen to three different types of loading.

Two separate types of tests were conducted to evaluate the behavior of the mechanical splices:

Monotonic Tension Test: The first test performed on all splice assemblies. The specimen was loaded from zero strain up through the 4 percent strain requirement and then on to failure. Each test was performed similar to the tension test specified in ASTM 370A for determining the yield strength of steel [ASTM 1999].

Stepped Cyclic Test: For this test the specimen was started at initial conditions of zero strain under zero load. The splice assembly was loaded in the same manner as in the monotonic load test but only until the average strain across the 20 bar diameter reached 2 percent. At this point in the test the load was reversed and the specimen was unloaded through zero load

The purpose of compressing the splice assembly was to insure that the splice itself underwent complete unloading in tension before being subjected to the next cycle of tension loading. After being unloaded, the assembly was then cycled four times under a tension load out to 2 percent strain. After this tension-to-compression cycle to 2 percent strain was completed four times, the testing was repeated again at strain values of 2.5, 3.0 and 3.5 percent strain. At each strain level, the testing cycle was conducted four times. After the final unloading at the 3.5 percent strain the assembly then loaded in tension out to failure.

20 assemblies were obtained from each manufacturer participant. These assemblies were divided into two groups. 12 were pulled monotonically to failure, 8 were cycled to 4 percent strain uniformly over 16 cycles and then pulled to failure.

The elongation was measured using Linear Differential Variable Transformers (LVDT's). Two LVDT's were placed at 180-degree intervals around the circumference of the mechanical connection assembly. The purpose of using three measurements taken at 180 degree intervals was to account for any initial lack of straightness of the connector/bar assembly and to insure accurate elongation measurements throughout the testing, both for monotonic and cyclic loading tests.

The two elongations were then averaged to determine the average overall elongation. This average elongation was divided by the original gage length of the splice/bar specimen to determine the average strain value for evaluation of the mechanical splice. Two other strain measurements were taken on the exterior of the mechanical splice itself and on the bar.

Testing was performed in the Structural Engineering Laboratory at the Anadolu University. Results were reported to the individual participating companies and were used to improve their individual systems as well as a means to develop new design requirements and policies within ACI and Turkish codes. Typical results are shown in Fig. 1. It can be seen that the results clearly indicate that the splice is capable of developing a large nonlinear strain in the bars and that system failure is above the 4% acceptance criteria. Fig 2 show similar results for the step cyclic testing regimes.

What is important in these figures is that the system presented as the “Company KT specimen” exhibited good overall system ductility with failures at strains well in excess of the 4% acceptance criterion. The step cyclic testing results show that the presence of cyclic loading did not reduce the failure ductility significantly and that this particular splice could indeed be expected to maintain load capacity well into the nonlinear regime.

Additional insight can be gained from study of the plots. These different readings permitted observation of where the strain was concentrated during testing. The data that was produced is interesting in that, as shown in Tables 2 and 3, the strain is not uniform across the splice. It can be seen in both of these figures that there tends to be a difference in strain readings in the splice body, as compared with the overall system. This difference shows where the more flexible parts of the connection are found and where the damage tends to be concentrated.

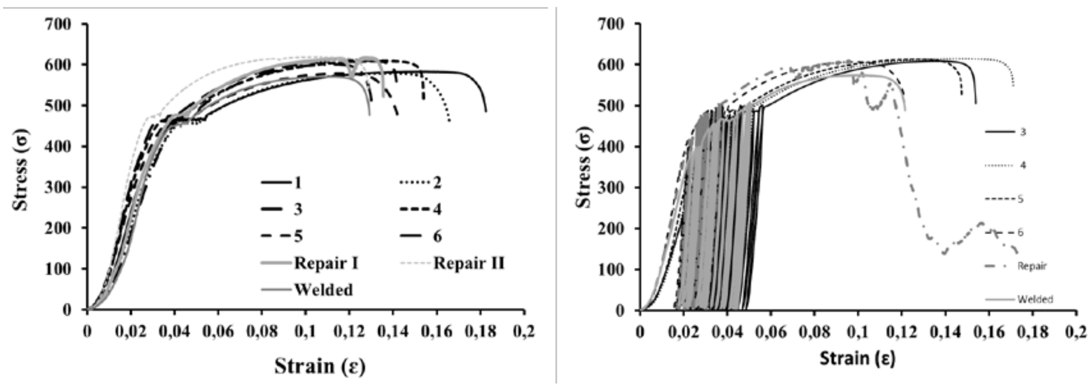

Figure 2. Stress-Strain diagram for Ø32 Bar Specimens_Monotonic and Stepped Cycling Tensile Test Test

Test Pattern	Yield σ_y MPa	Ultimate σ_U Mpa	ε_U (%)	σ_U / σ_y	Slip ≤ 1mm	Ductility ≥%70 ε_U	PASS 1.25 f_y	PASS 4%	PASS test
Monotonic Tensile Test	457*	582	17	1.27	-	11.9	Y	Y	Y
	455	580	16.5	1.27	0	Y	Y	Y	Y
	462	610	14	1.32	0	Y	Y	Y	Y
	460	608	15.4	1.32	0	Y	Y	Y	Y
	465	578	14.1	1.25	0	Y	Y	Y	Y
	465	600	13	1.29	0	Y	Y	Y	Y
	468 Repair I	615	12	1.31	-	N	Y	N	N
	469 Repair II	618	12.7	1.32	0	Y	Y	Y	Y
	450 Welded	570	12.9	1.27	0	Y	Y	Y	Y

Table 2 Summary of company Ø32 Bar Specimens(Tensile Tests)

Table 3. Summary of company Ø32 Bar Specimens (Stepped CyclicTensile Tests)

4 CONCLUSION

The purpose of this research was to provide recommendations for use of mechanical connection systems in both static and seismic conditions.

The research was motivated by the facts that the current criteria in Turkish reinforced concrete standard, TS 500were based on no research, while there had been advancement in this field worldwide in recent years.

New criteria can be used on mechanical connection systems in both static and seismic conditions and also proposed for the upcoming review of Turkish code.

Two criteria were proposed for use of mechanical connection systems in seismic conditions,being strength and serviceability limit state.

Preliminary tests indicated that mechanical connection systems commonly used in Turkey met the proposed criteria.

The results of this testing program at the Anadolu University have shown that mechanical splices are indeed capable of effectively connecting reinforcing bars together and that many are capable of producing bar breakage as the failure mode; thus indicating that the splice is stronger than the parent bar itself. The problem is that not all splices that are available will produce this kind of performance. Moreover, not all splices need to exhibit this type of capacity. Indeed many applications are such that performance to a lower level is acceptable. At present in the ACI 318 (1999), there are two levels of splice performance that are permitted – nonseismic and seismic levels. The difference is the strength (and ductility) of the splice and where it is permitted in the structure. The challenge that exists in the future is to provide robust, but cost effective, means to identify the mechanical splices that meet the higher seismic category of performance. The results of this study, while not conclusive in themselves, show that steps are being taken towards this end. A consistent performance definition and testing protocol for mechanical splices that will allow the designer's vision to be upheld in these important connection regions is within our grasp.

Test Pattern	Yield σ_y MPa	Ultimate σ_U Mpa	ε_U (%)	σ_U / σ_y	Slip ≤ 1mm	Ductility ≥%70 εU	PASS 1.25 f_y	4%	test
Stepped Cyclic Tensile Test	461*	609	15.4	1.32	0	Y	Y	Y	Y
	462	614	17.0	1.31	0	Y	Y	Y	Y
	465	613	14.7	1.31	0	Y	Y	Y	Y
	467	607	12.1	1.30	0	Y	Y	Y	Y
	467 Rerpair I	608	-	1.30	-	N	Y	N	N
	461 Welded	576	12.1	1.25	0	Y	Y	Y	Y

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