THE PERFORMANCE OF MECHANICAL SPLICES

Steven L MCCABE[1]

SUMMARY

This paper will present the results of the largest independent study of mechanical reinforcement splices conducted at the University of Kansas of all splicing systems marketed in the United States. The results show that many of the commercially available splices were capable of developing the full ultimate strength of the bar. Others were able to develop the yield strength of the bar and significant amounts of strain across the connection. A third group was able to develop only minimal levels of strain. All the splicing systems tested were able to meet the existing strength requirements; however, the level of strain developed was widely variable. The results of this study and related discussion within the engineering community were responsible for a significant change in attitude in the US. Several governmental agencies, notably Caltrans, began requiring seismic qualification of all mechanical splices as a result of these findings.

INTRODUCTION

The design of reinforced concrete structures is based on several basic assumptions and tenants. Among these fundamental concepts is that reinforcement will be provided to resist any tension that may be present in the member. Moreover, it is assumed that the reinforcement is anchored – that is, developed -- and continuous in the regions where it is shown by analysis to be required.

This rather simple statement causes significant challenges for designers. Not only must analysis be performed to determine the amount of steel required, but the required development lengths for adequate bond of the reinforcing steel also must be computed. Lastly all of this information is put to use in the detailing of the section. In many cases there may be problems locating all of the steel required within the space provided. If large amounts of steel are present, as found in seismic regions, the detailer is faced with a three-dimensional interference puzzle that must be solved while meeting building code requirements for spacing and cover.

Moreover, reinforcing bars are rolled in set lengths, in the US typically 12.3 m (40 feet). Thus, in many applications, the designer or detailer must provide splices for the reinforcing bars. In most situations where splices of reinforcement are involved, the splice of choice is the lap splice. These splices are straightforward in concept with the length of lap assumed to be directly related to the development length. However, there may be significant problems in locating the splices while still meeting all of the code requirements. In addition, there may be additional cover or confinement required to control the lap length so as to reach the capacity needed in the length provided. The splicing of large diameter bars can lead to required lengths easily over a meter in length, a requirement that is not easily met in many situations. Furthermore, the splicing of large column bars, Nos. 44 and 57 bars (44 and 57 mm in nominal diameter, respectively) in the US bar series, is typically not allowed by the American Concrete Institute 318 Building Code [ACI 1995].

If the splice cannot be placed in the section area provided, or if the bar being spliced is not adequately developed in the splice length provided, then the designer must look at alternatives. Choices include redesigning the splice to provide additional confinement to reduce the splice length, repositioning the splice or even changing the bar size.

[1] *Department of Civil and Environmental Engineering ; Unversity of Kansas; USA*

The designer, however, increasingly may look at other alternatives where the splice technology itself may be changed, and the lap splice replaced with either a welded splice or a mechanical splice. Welded splices represent a special challenge. In the United States, the designer must be aware that the welding of typical new billet ASTM A 615 bars must be approached cautiously since essentially no chemistry control exists on the production of these bars [ASTM 1999]. Special tests and welding procedures may be required to produce a ductile weld [AWS 1992]. Cost also is a consideration.

This leads to mechanical splicing as the primary alternative to lapped splices used by designers in the United States. The manufacturers of these splices assert that when properly installed, mechanical splices offer performance that can be equal to or better than that of lapped bars.

This paper will present the results of the largest independent study of mechanical splices conducted in the United States and is specially aimed at the performance issue. The work was conducted at the University of Kansas in response to a question raised by the ACI Building Code Seismic Subcommittee as to the viability of the present design requirements for splices subjected to seismic loading.

CODE REQUIREMENTS FOR MECHANICAL SPLICES

Mechanical splices are unique in that there are no industry standards in place in the United States, nor are there any calculations that the engineer can perform to assure themselves that the splice can adequately develop the load required. Instead the splice manufacturer is in the position to guarantee that the splice will perform in the manner proscribed by the code bodies involved. 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 are federal 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. 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.

There are other requirements that have been defined to prequalify splices for use in projects. The International Conference of Building Officials (ICBO), publishers of the Uniform Building Code, has an Evaluation Service that developed their own requirements for mechanical splices that requires testing of splices to failure. Monotonic tension and compression tests plus a complex cyclic test regime is required to evaluate splices submitted by manufactures for approval for use in structures designed using the Uniform Building Code [ICBO ES 1998]. Moreover, the California Department of Transportation (Caltrans) adopted its own testing procedure for the prequalification of splices. The Hibernia oil platform located off the east coast of Newfoundland, Canada was constructed using no lapped splices at all. Instead over 2 million mechanical splices were used that were qualified using yet another testing procedure, the ACI 439 Test Procedure. The application of this testing procedure during a research program at the University of Kansas (KU) is the subject of this paper.

TESTING PROGRAM

The testing program was conducted at KU from 1992-1997 to ascertain how well typical commercial quality mechanical splice systems as sold in the United States would perform under simulated seismic loading. This program was commissioned by American Concrete Institute (ACI) Committee 439, Steel Reinforcement, in response to an inquiry by the Seismic Subcommittee of the ACI Building Code Committee, ACI 318-H. The

ACI 318 provisions in the 1992 and 1995 editions required simply that the coupled system meet 125 percent of the nominal yield of the bar. This requirement was intended to satisfy the situation where overstrength bars were coupled by the mechanical system -- a situation typically found in practice. This is especially true when splicing ASTM A 615 bars where there is no maximum value of bar yield or tensile strength and also no control on carbon content; the bar may easily exceed the minimum yield of 420 MPa by 25% or even more.

The question of how the mechanical splice systems would perform under severe cyclic loading, such as lateral loading from earthquake, in which perhaps significant inelastic demands are made on members and connections is an entirely different consideration. In these cases, the ability to maintain load capacity well into the inelastic regime is the issue and may not be satisfied by strength-only criteria. The research effort at KU was a direct result of that inquiry and was intended to answer the question posed by ACI Subcommittee 318-H.

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 25-mm in diameter and to use only Grade 420 steel with a nominal yield of 420 MPa. New billet carbon steel conforming to ASTM A 615 was used for all tests. Low-alloy ASTM A 706 steel would be a better selection from a material property standpoint [ASTM 1999]; however, in the US today most structures are constructed using A 615 reinforcing steel.

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 20 bar diameters in length, or approximately 500 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. A strain level of 4% as measured across the 20 bar diameter gage length was used as an acceptance criterion.

Three 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]. Load/Strain rates were maintained within ASTM 370's specification parameters using a load rate between 0.45 kN /min. (0.1 kips/min.) as a minimum and 445 kN/min. (100 kips/min.) as a maximum. The target load rate was maintained at approximately 265 kN/min. (60 kips/minute).

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 to a target compression load of 44.5 kN (10 kips) or 87.5 MPa (12.7 ksi). 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. The target compressive load of 44 kN was to ensure that a failure due to buckling did not occur while putting a sufficient compressive load on the splice so as to "click" the splice from tension into compression. After being compressed, 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.

Uniform Cyclic Test: In this test the splice assembly was loaded from zero strain at zero load out to a full 4 percent strain on the first load cycle. Upon reaching the 4 percent strain value the assembly was unloaded back through zero load to a target compression load of 44.5 kN (10 kips) or 87.5 MPa. This cycle was completed a total of sixteen times. After the sixteenth cycle the specimen was then loaded in tension to failure.

Accordingly, participation was solicited from industry and No. 25 (25-mm diameter) bar coupled assemblies were supplied to the University of Kansas Structural Engineering and Materials Laboratory. Participating domestic manufactures included Richmond Screw Anchor, Erico, Dywidag Systems International, Barsplice Products, Splice Sleeve North America, Dayton Superior, Williams Form Engineering, Barlock Industries and Rebar CouplerBox as well as two European splice manufacturers, CCL-Ltd. and Metalock Industries.

Nine assemblies were obtained from each manufacturer participant. These assemblies were divided into three groups. Three were pulled monotonically to failure, three were cycled to 4 percent strain uniformly over 16 cycles and then pulled to failure. The third group of three specimens were step cycled and following the 16th complete cycle, the splice assembly was pulled to failure.

The reinforcing steel supplied by Birmingham Steel was No 25 bars made of Grade 420 A615 steel from a single source and heat. The reinforcing steel was selected to be as strong as possible but with an actual yield strength not to exceed 535 MPa (78 ksi) and was intended to perform so that the strength and ductility demands on the mechanical connections would be as severe as possible.

All testing was performed on a servohydraulic test system manufactured by the Instron Corporation. The system, Model 1334, had a load rating of ± 450 kN (100 kips) with a stroke range of ± 125 mm (5.0 in.). For all testing, the system was used in the 100% load range and 50% (± 63.5 mm [2.5 in.]) stroke range and load was read from the machine load cell.

The primary strain measurement to be considered was a 20 bar diameter gage length, which encompasses both the mechanical splice and the two bars. The use of 20 bar diameters as a gage length was a decision based on discussions within ACI. In a typical beam with No. 25 longitudinal reinforcing bars, the 508 mm (20-in.) gage length was intended to represent the distance over which the effect of any stiffness or softness (compared to the reinforcing bar) of the mechanical connection could be considered to be spread out relative to structural response.

The strain specified to be measured for determination of the mechanical splices adequacy for seismic use was the average strain measured across the entire 20 bar diameter gage length. The elongation was measured using Linear Differential Variable Transformers (LVDT's). Three LVDT's were placed at 120-degree intervals around the circumference of the mechanical connection assembly. The purpose of using three measurements taken at 120 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 three 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. Micro extensometers were used to take these measurements. These strain readings could be plotted against the corresponding load and compared to the LVDT strain across the 20 bar diameter gage length [McCabe et al 1998].

TEST RESULTS

Testing was performed in the Structural Engineering and Materials Laboratory at the University of Kansas. 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. The identity of the participating companies was held confidential as to the specific results involved. Results included load and elongation data pairs that were recorded electronically as well as manual data taken by measuring the elongation of the splice specimen and comparing values to the initial lengths; this data was used to verify the validity of the electronic data [McCabe et al 1998].

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. Figures 2 and 3 show similar results for the two cyclic testing regimes. The results show that the cyclic testing did not alter the performance of the specimens significantly – probably a result of the lack of full reversed cyclic loading. Reversals were not possible with the in-air tests since stability of the specimen was a strong consideration given the specimen length used.

What is important in these figures is that the system presented as the "Company L specimen" exhibited good overall system ductility with failures at strains well in excess of the 4% acceptance criterion. The two 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. In comparison several other systems tested failed at strain levels of 1%, exhibiting relatively low ductility that is well below the acceptance criteria.

Additional insight can be gained from study of the plots. As mentioned above, the strain readings that were taken consisted of measuring the strain across the 20 bar diameter gage length as well as constituent readings on the splice body itself and on the bar itself. 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 Figs. 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.

The overall result of the testing revealed that some systems tended to produce large amounts of slippage in the connection, while others exhibited actual elongation of the splice body itself. Another group tended to focus the strain into the bar itself and away from the splice; these systems tended to produce the best overall performance. Care in the design and fabrication of the connection *mechanism* within the splice appears to be the key to improve performance.

Another observation is that in general, the performance of these systems was adequately depicted by monotonic testing. Systems that were nonductile were discovered during monotonic testing. Only in limited cases did splice performance under cyclic loading significantly change. Thus, a straightforward monotonic test will indeed identify most of the splices where there may be performance questions, within a limited testing budget. That is not to say that cyclic testing does not have its place. There are many applications where a testing regime such as the ACI 439 test will identify degradation of the connection with cycles. This is more likely to be the case where connections are a gripping type of connection or one with grout.

Another question is the use of the 20 bar diameter gage length. This choice was made to simulate the action of a plastic hinge where strain would be distributed over a region of the member, typically assumed to be the effective depth of the member. This selection was based on an overall view of the performance required of the spliced bar. ICBO enforced the use of short specimens though the requirement of large compression levels in the splice under cyclic loading. Short specimens also are being used at this time by Caltrans in their testing procedures. In these situations, the overall performance is less of an issue than the strain that can be developed in the splice connection. These systems utilize a gage length of on the order of the splice plus two bar diameters. Thus the performance is limited to the connection itself.

OBSERVATIONS ON PERFORMANCE

The overall results can be divided into three distinct groups: (1) splices capable of developing large strains leading to failure by bar breakage; (2) splices that exhibited system ductility of at least 3% and were able to yield the bar leading to failure in the bar or at the bar-splice connection; and (3) those developing strength but with limited ductility, falling below 3% system strain. It is interesting to note is that nearly all of the specimens tested met the present strength criterion of 125% of nominal yield. Yet when loaded monotonically, some specimens consistently failed at system strains that were well below 3%, with some even below 1.5% strain. Thus, a designer faced with utilizing mechanical splices in a critical location in a structure must not rely solely on strength criteria alone but must ask the supplier for performance information regarding how well the splice performs under severe loading. This data is available but is generally not reported in the literature provided with these types of products. The designer must be aware of the concept of ductility and the need for the splice to load capacity over a wide strain range. They must ask appropriate questions and receive documentation as to the total performance.

Furthermore, building code bodies need to specify both strength and ductility levels that are consistent with the design philosophy present in other portions of their code. Design rules that address the required performance of constituent materials, as well as the performance of members and systems designed from these materials must be comprehensive. Failure to address the performance of the splices between individual reinforcing bars can be a serious oversight and can lead to unexpected performance problems.

CONCLUSIONS

The use of mechanical splices as an alternative to lapped splices is a viable one in many situations. Clearly the advantages of effective splicing and the reduction in space taken up by the splice are significant and partially outweigh the cost of the element itself and its installation. The key question comes down to performance.

The results of this testing program at the University of Kansas 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 318-99 [ACI 1999] Building Code 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.

BIBLIOGRAPHY

American Association of State Highway Transportation Officials-- AASHTO (1996), *Design Specification for Highway Bridges*, 16th Edition, Washington D.C.

American Concrete Institute - ACI Committee 318 (1995), *Building Code Requirements for Structural Concrete* (ACI 318-95) and *Commentary – ACI 318R-95*, American Concrete Institute, Farmington Hills, MI.

American Concrete Institute - ACI Committee 318 (1999), *Building Code Requirements for Structural Concrete (ACI 318-99)* and *Commentary – ACI 318R-99*, American Concrete Institute, Farmington Hills, MI.

American Concrete Institute - ACI Committee 439 (1998), *Standard Specification for Mechanical Reinforcement Splices for Seismic Designs using Energy Dissipation Criteria (ACI-439 XX),* under review by ACI Technical Activities Committee.

American Society for Testing and Materials (ASTM) (1999), "Standard Test Methods and Definitions for Mechanical Testing of Steel Products (A 370 - 97a)," *1999 Annual Book of ASTM Standards, V. 4.02,* American Society of Testing and Materials, West Conshohocken, PA, pp. 166-211.

American Society for Testing and Materials (ASTM) (1999), "Standard Specification for Billet Steel Bars for Reinforcement of Concrete (A 615/A 615 M - 96a)," *1999 Annual Book of ASTM Standards, V. 4.02,* American Society of Testing and Materials, West Conshohocken, PA, pp. 308-316.

American Society for Testing and Materials (ASTM) (1999), " Standard Specification for Alloy Steel Bars for Reinforcement of Concrete (A 706/A 706 M - 96b)," *1999 Annual Book of ASTM Standards, V. 4.02,* American Society of Testing and Materials, West Conshohocken, PA, pp. 346-350.

American Welding Society (1992), *Structural Welding Code – Reinforcing Steel, D1.4-92,* Miami.

Building Officials and Code Administrators International (1999), *Building Code*, BOCA International, Country Club Hills, IL.

International Conference of Building Officials (1997), *Uniform Building Code*, Whittier, CA.

International Conference of Building Officials Evaluation Service (1998), "Acceptance Criteria for Mechanical Connectors for Steel Bar Reinforcement," *Acceptance Criteria 133 (AC 133)*, Whittier, CA.

McCabe, S.L. D. Schlimme, J.P. Jones and B. Ge (1998), "The Performance of Mechanical Reinforcing Steel Splices under Monotonic and Cyclic Loading," Vols. I-III, *SM Report No. 53,* University of Kansas Center for Research, Inc., Lawrence, Kansas, 830 pp.

Southern Building Code Congress International (1997), *Standard Building Code*, Birmingham, AL.

Table 1 Summary of Company L No. 25 Bar Specimens

TEST PATTERN	LOAD		STRESS			STRAIN (%)				Pass		
							Mark-of-the-bar					
	Kips	kN	ksi	MPa	fy	LVDT	Top	Bottom	Splice	1.25fy	4%	test
Monotonic tensile Test	72.2	320.9	91.9	633.7	1.53	9.77	11.40	23.20	0	Y	Y	Y*
	71.3	317.2	90.8	626.2	1.51	11.80	8.30	12.20	0	Y	Y	Y*
	77.5	344.7	98.7	680.7	1.65	10.40	10.40	17.20	0	Y	Y	Y*
	72.4	322.1	92.2	635.9	1.54	8.70	11.40	12.60	0	Y	Y	Y*
	73.2	325.6	93.2	642.9	1.55	7.80	14.40	7.90	0	Y	Y	Y*
Average	73.3	326.1	93.3	643.9	1.56	9.69	11.18	14.62	0			
							12.90					
Step Cyclic Test	75.0	333.6	95.5	658.7	1.59	9.17	16.40	13.19	0	Y	Y	Y*
	73.3	326.1	93.4	643.8	1.56	8.36	12.80	10.40	0	Y	Y	Y*
	76.1	338.5	96.9	668.4	1.62	10.00	10.60	16.90	0	Y	Y	Y*
	72.5	322.5	92.4	636.8	1.54	9.21	24.60	8.60	0	Y	Y	Y*
	73.1	325.2	93.1	642.0	1.55	9.60	16.20	11.90	0	Y	Y	Y*
	68.2	303.4	86.9	599.0	1.45	10.60	13.80	12.90	0	Y	Y	Y*
Average	73.0	324.9	93.0	641.5	1.55	9.49	15.73	12.32	0			
							14.02					
4% Cyclic Test	73.7	327.8	93.89	647.3	1.56	8.20	23.90	7.70	0	Y	Y	Y*
	71.3	317.2	90.83	626.2	1.51	8.09	15.00	9.90	0	Y	Y	Y*
	76.2	339.0	97.07	669.3	1.62	6.40	9.50	21.70	0	Y	Y	Y*
	70.8	314.9	90.19	621.8	1.50	6.80	16.20	11.80	0	Y	Y	Y*
	77.8	346.1	99.11	683.3	1.65	10.10	9.60	19.30	0	Y	Y	Y*
	75.7	336.7	96.43	664.9	1.61	11.60	15.00	17.00	0	Y	Y	Y*
Average	74.3	303.3	94.6	652.1	1.58	8.53	14.87	14.57	0			
							14.72					

* fracture on reinforcing bar.

LOAD vs STRAIN for MONOTONIC TEST

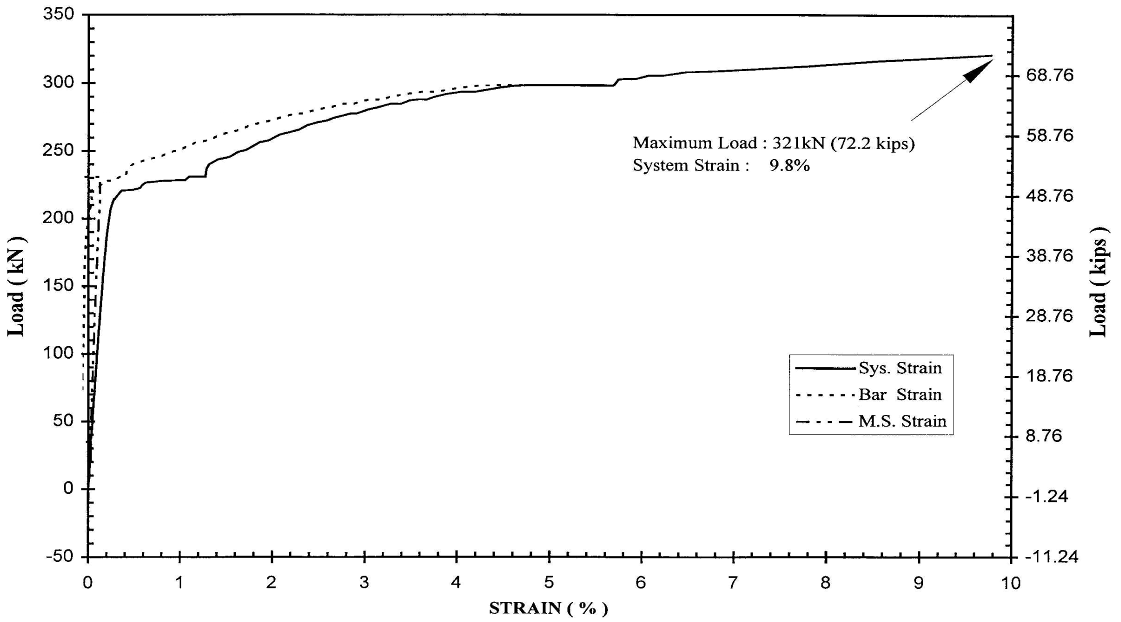

Fig.1 Monotonic Test Results for Company L

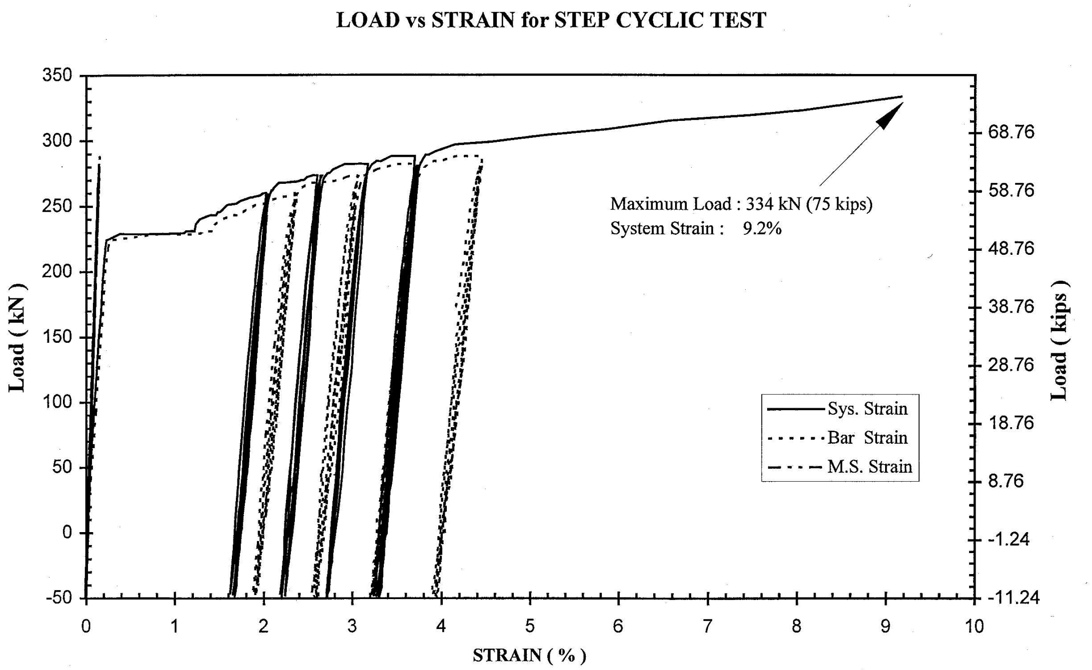

Fig. 2 Stepped Cyclic Test Results for Company L Specimen

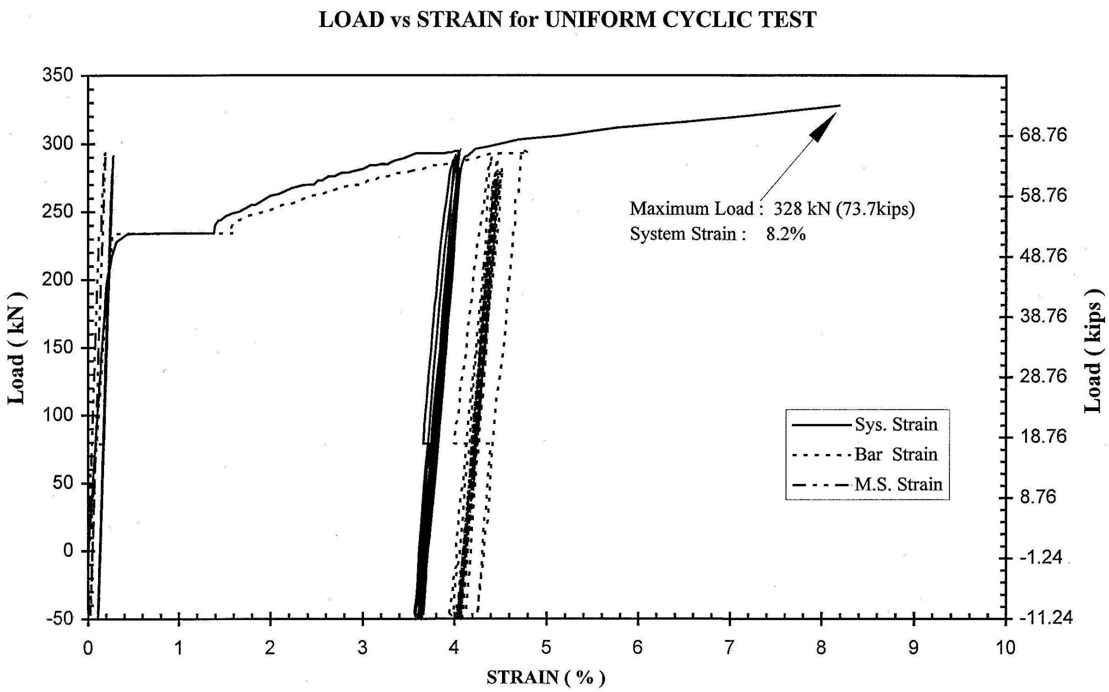

Fig. 3 Uniform Cyclic Test Results for Company L Specimen