TOWARDS EARTHQUAKE-RESISTANT CONCRETE STRUCTURES WITH ULTRA HIGH-STRENGTH STEEL REINFORCEMENT

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ABSTRACT:

New design provisions for structural concrete (ACI 318-08) allow the replacement of lower strength steels (e.g., ASTM A615 and ASTM A706) with steels having specified yield strengths well in excess of 80 ksi (550 MPa). Since the use of higher yield strength steel in member design is allowed only when determining required confinement reinforcement in compression members, and not when determining reinforcement required for shear, torsion, and/or flexure, the full potential for steel reduction is severely restricted. A collaborative experimental program between Penn State University and Purdue University is aimed at studying the behavior of concrete members reinforced both longitudinally and transversely with ultra high-strength steels. The investigation focuses on the deformation capacity of beams and columns subjected to displacement reversals and on corrective measures for achieving the rotation capacities associated with traditional reinforcing steels. A summary of the experimental program is presented with details on test setup, loading protocol, number of specimens, and specimen geometry.

KEYWORDS: Advanced high-strength steel, cyclic loading, deformation capacity, high-performance.

1. INTRODUCTION

Increasing demands of the new millennium for sustainable and durable structures, and the limited available resources, have awaken the need for newer construction technologies and efficient use of structural materials. With the growing number of tall buildings and mammoth structures being built in the near future, the architecture, engineering, and construction (AEC) industry calls for the use of more efficient materials.

For many years the design of reinforced concrete structures in the U.S. has been dominated by the use of steel reinforcement with a specified yield strength, f_y, of 60 ksi (410 MPa). Although higher values of f_y are allowed, the f_y has been limited to 80 ksi (550 MPa) since the 1971 edition of ACI 318[1]. The recently completed ACI 318-08[2] permits the use of reinforcement with f_y limited to the stress that corresponds to a strain of 0.35% but not to exceed 80 ksi (550 MPa) except for transverse reinforcement where a yield strength of 100 ksi (690 MPa) is allowed. The exception applies to requirements for confinement in compression members and not to requirements for shear, torsion, and/or flexure-and-axial strength.

Steel with yield strength in excess of 80 ksi (550 MPa) is currently commercially available in the U.S. and it is often called Advanced High-Strength Steel[3] (AHSS) or Ultra High-Strength Steel[4] (UHSS). In this paper, the term UHSS is used to designate a steel having yield strength in excess of 80 ksi (550 MPa) and elongation at rupture of 5% or more. This designation is adopted regardless of the microalloy elements used for strengthening of the steel. After the approval of ASTM A1035[5], there has been growing interest in UHSS bars; however, there is paucity of test data on the behavior of concrete members reinforced with UHSS bars, especially when used as main longitudinal reinforcement.

Using UHSS introduces several benefits to the AEC industry. By using higher strength bars, required member cross sections and reinforcement quantities may be reduced leading to savings in material, shipping, and placing costs. Also, the reduced number of bars prevents congestion problems leading to better quality of construction.

2. RESEARCH BACKGROUND

New provisions in the 2008 edition of the ACI Building Code, ACI 318-08[2], allow the replacement of lower-strength steels (e.g. ASTM A615 and A706) with ASTM A1035 bars having yield strengths of up to 100 ksi (690 MPa). The changes are based on tests of concrete columns where UHSS was used only as transverse reinforcement[6,7,8]. Because the use of higher yield strength in member design is allowed only when determining required confinement in compression members, and not when determining required shear and/or compression-and-flexure reinforcement, the potential for steel reduction is limited.
Representative tensile properties of conventional high-strength steel commonly used in the U.S. (ASTM A615 and A706) are shown in Fig. 1 next to a few samples of commercially available UHSS deformed bars. The elongations at rupture shown in Fig. 1 correspond to the minimum prescribed by their denomination.

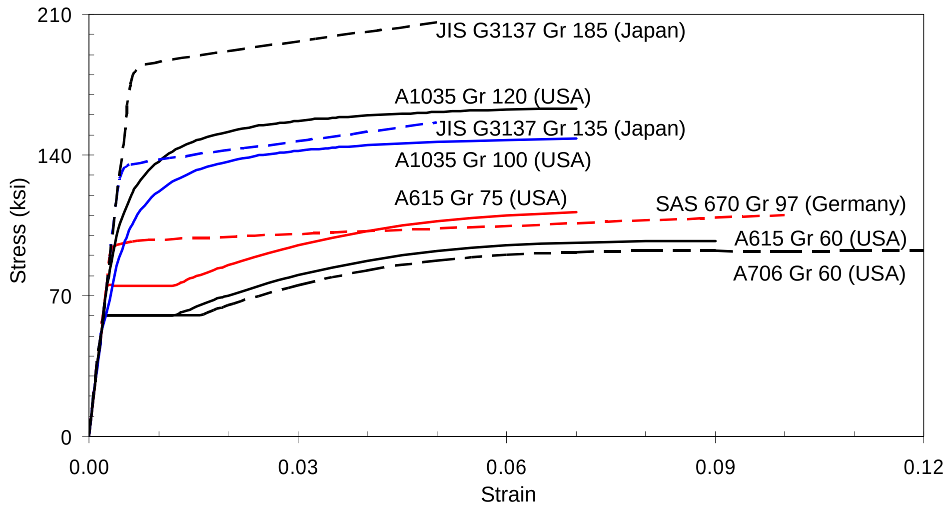

Figure 1 – Tensile Properties of Conventional Deformed Steel Bars (Gr 60 and 75) Compared to Ultra High-Strength Deformed Steel Bars (Gr 97 and above).

The limits on the specified yield strength of reinforcing bars in the ACI 318 Code trace back to its 1963 edition[9] and are primarily related to the prescribed limit on compressive strain of 0.003 for concrete and to control of crack width at service load.
The ACI 318-63 Code[9] introduced strength design provisions with restrictions on the yield strength of nonprestressing steel. In Section 1505, ACI 318-63 specified an upper limit of 75 ksi (520 MPa) on the yield strength of compression reinforcement and 60 ksi (420 MPa) for tension reinforcement (unless special tests satisfied crack control requirements, in which case 75 ksi (520 MPa) was permitted). The Commentary[10] to ACI 318-63 states:

> "High strength steels frequently have a strain at yield strength or yield point in excess of the 0.003 assumed for the concrete at ultimate. The requirements of Section 1505 are to adjust to this condition… The maximum stress in tension of 60,000 psi without test is to control cracking".

In the 1971 edition of ACI 318[1], the limit on the specified yield strength of reinforcement (other than prestressing steel) was increased to 80 ksi (550 MPa) in order to continue accommodating the highest yield strength covered by contemporary ASTM standards. Tension reinforcement, with design yield strength in excess of 60 ksi (420 MPa), no longer triggered special testing procedures. The revised limits adopted by ACI 318-71 effectively incorporated the findings from the extensive laboratory investigations reported in the 1960s by the eight-part report of the Portland Cement Association (PCA)[11,12] and by Todeschini[13]. These studies were essentially an extension of the comprehensive column investigation initiated in 1929 by Richart and Brown[14] under the direction of ACI Committee 105. These early efforts used longitudinal steel bars with an actual yield point ranging from 55 ksi (380 MPa) to 120 ksi (830 MPa) where the steel had, typically, elongations at rupture exceeding 5%. A summary of the findings contained in these studies[11-14] is beyond the scope of this paper but two conclusions are worth emphasizing: (1) in flexural members, "both maximum and average crack width are essentially proportional to reinforcing steel stress"[12], and (2) "for steels with flat yield plateaus up to approximately 90,000 psi, the yield stress can be developed"[13] in tied columns.

A major effort in the 1980s and 1990s was led by Japanese researchers on the use of ductile reinforcing bars with nominal yield stress in excess of 80 ksi (550 MPa) for transverse and/or longitudinal reinforcement. A summary of these efforts was presented by Otani, et al.[15] with a view on initial stiffness calculations and limited to beam specimens having the same amount of top and bottom longitudinal reinforcement. The test data of 105 beams presented by Otani et al. suggest that UHSS is commonplace in Japanese construction.

A strategy for taking advantage of UHSS bars when used as primary reinforcement is to control the crack width and to enhance the usable compressive strain of concrete. High tensile and compressive strain capacities are attainable by concrete reinforced with dispersed fibers. A new generation of high-performance fiber-reinforced cementitious composites (HPFRC) includes formulations of fibers and matrices to achieve strain hardening behavior with fiber contents of 1.5% by volume in concrete with an aggregate size of up to ½ in. (13 mm)[16].

Although practical evidence exists on the benefits of incorporating UHSS as transverse reinforcement and as longitudinal reinforcement in concrete members, experiments are needed for exploring the potential benefits provided by the addition of engineered fibers into the concrete mix in order to make effective use of rebar with yield strengths of unprecedented high values.

Studies on the compressive properties of cementitious composites have shown that the introduction of fibers into the matrix delays spalling of the cover and increases the load capacity and the ductility of columns over that of comparable non-fiber-reinforced specimens[17]. Several empirical relationships are available for the compressive stress-strain curves of fiber-reinforced concrete (FRC). A few of these are plotted in Fig. 2 to show that FRC in compression exhibits an increase in the strain at the peak stress and substantially higher ductility and toughness as compared to plain, cement-based matrices[18-21].

Several research projects have explored the application of fiber-reinforced composites in earthquake-resistant design as summarized by Parra-Montesinos[22]. Results from these studies revealed HPFRC to be effective in increasing shear strength, displacement capacity, and damage tolerance in members subjected to large inelastic deformations, but none of these studies incorporated UHSS reinforcement.

More recently there has been a revival of research on UHSS as documented in the nearly completed design guidelines[23] for the use of ASTM A1035[5]. However, this effort limits the design stress of steel in compression to 80 ksi, does not incorporate the effects of fibers, and does not cover applications of steel in tension with yield in excess of 100 ksi. To date, there continues to be no evidence of test data on columns with non-prestressed ductile UHSS, in combination with FRC or HPFRC, subjected to either monotonic or reversed-cyclic loading. The experimental program described herein will address this void.

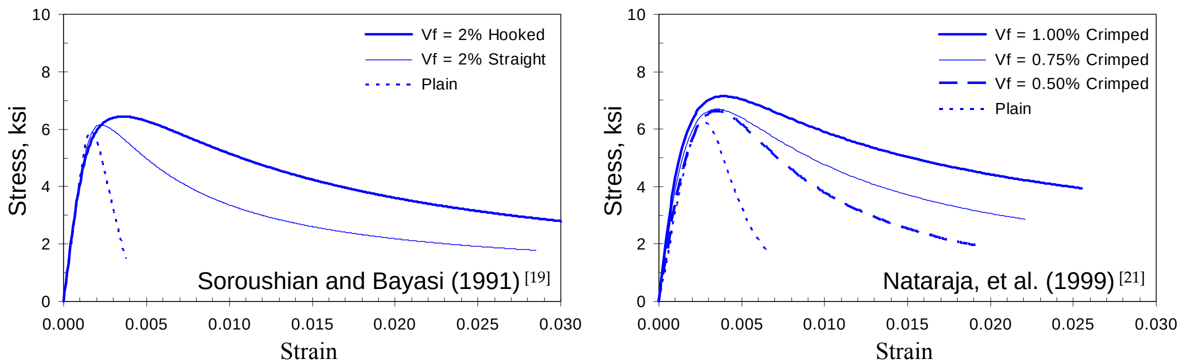

Figure 2 – Typical Compressive Stress-Strain Curves for Steel-FRC, Fiber L/d of ~75.

3. RESEARCH SIGNIFICANCE

The use of UHSS bars in conventional reinforced concrete has been limited due to the paucity of experimental data, the lack of design guidelines, and the severe restrictions in existing building codes. The deformability of conventional concrete both in tension and compression needs to be enhanced in order to make effective use of the unprecedented high yield strength and elongation of newly developed UHSS reinforcing bars.
A strategy for taking advantage of UHSS in the immediate future is to use high-performance fiber-reinforced concrete (HPFRC) characterized by a tensile stress-strain response that exhibits strain hardening accompanied by multiple cracking[16,22]. HPFRC allows high tensile stresses in primary longitudinal steel bars while controlling the width of cracks in the surrounding concrete. In addition, HPFRC significantly increases the usable compressive strain to values in excess of 0.01 or more than 3 times the value associated with conventional concrete (see Fig. 2 for a volume fraction, V_f, of 2% hooked). The experimental program described herein explores this novel concept.

4. EXPERIMENTAL PROGRAM

4.1. Beam Tests

To evaluate the flexural behavior of concrete beams reinforced with UHSS bars, a series of experiments has been designed at Penn State University. The purpose of this series of tests is to evaluate the deformation capacity of such beams under cyclic and monotonic loading. Table 4.1 shows a summary description of the beam specimens to consider. The main variables of the beam testing program include:

- Yield strength of longitudinal reinforcement, f_y = 60 and 100 ksi [410 and 690 MPa]
- Steel fiber-reinforced concrete (V_f = 0% and 1.5%)
- Spacing of transverse reinforcement ($s = d/4$ and $d/2$)
- Loading type (cyclic and monotonic)
- Ratio of compression reinforcement to tension reinforcement (ρ'/ρ = 0.5 and 1.0)

Table 4.1 Summary of Proposed Beam Tests (Penn State University)

#	Specimen[a]	Longitudinal			Transverse (2#3)		M_{pr} [b]	V_{pr} [c]	V_s [d]	$V_{pr} - V_s \geq 0$
		Top	Bottom	f_y, ksi	Spacing, in	f_y, ksi	Kip-ft	$b.d\sqrt{f'_c}$	$b.d\sqrt{f'_c}$	$b.d\sqrt{f'_c}$
1	CC4-X	4#7	4#7	60	2	60	105	5.3	5.3	0.0
2	UC4-X	4#6	4#6	100	2	60	110	5.5	5.3	~0.0
3	UC4-F	4#6	4#6	100	2	60	110	5.5	5.3	~0.0
4	UC2-F	4#6	4#6	100	4	60	110	5.5	2.7	2.8
5	CC4-XS	4#7	4#7[e]	60	2	60	105	5.3	5.3	0.0
6	CC4-X$	2#7	4#7[e]	60	2	60	105	5.3	5.3	0.0
7	UC2-FS	4#6	4#6[e]	100	4	60	110	5.5	2.7	2.8
8	UC2-F$	2#6	4#6[e]	100	4	60	110	5.5	2.7	2.8

[a] Specimens are subjected to cyclical loading unless noted otherwise. Specimen identification is based on the following symbols:
CC: Conventional longitudinal and Conventional transverse reinforcement.
UC: Ultra high-strength steel longitudinal reinforcement and Conventional transverse reinforcement.
UU: Ultra high-strength steel longitudinal reinforcement and Ultra high-strength steel transverse reinforcement.
4: $d/s \approx 4$ where d is effective depth and s is spacing of transverse reinforcement.
2: $d/s \approx 2$.
X: reinforced concrete without fibers, or fibers eXcluded.
F: steel Fiber-reinforced concrete, 1.5% volume fraction, fiber length-to-diameter ratio of 80.
S: specimen with Symmetrical reinforcement under monotonic loading.
$: specimen with a$ymmetrical reinforcement under monotonic loading.
[b] M_{pr} is the probable moment calculated for a concrete compressive strain of $\varepsilon_u = 0.003$ and a steel tensile stress of $1.25\,A_s \,.\, f_y$.
[c] V_{pr} is the shear associated with M_{pr} in terms of $b.d\sqrt{f'_c}$ [lb,in.], where b and d are the member width and effective depth, f'_c = 6,000 psi.
[d] V_s is the shear strength attributed to the transverse steel reinforcement, A_v, determined by $V_s = A_v \,.\, f_y \,.\, d/s$.
[e] For the specimens subjected to monotonic loading, the bottom reinforcement is in tension.

The value of V_{pr} presented in Table 4.1 is the shear strength corresponding to the probable moment resistance, M_{pr}, calculated without a strength reduction factor ($\phi = 1.0$) and replacing f_y with $1.25f_y$. Values of f_y correspond to the nominal yield strength of reinforcement. The target concrete strength is 6,000 psi [41 MPa].
Each specimen consists of two cantilever beams connected by a central stub. The geometry of a typical specimen is shown in Fig. 3. The modules are going to be loaded through the central stub in single curvature. The shear span to effective depth ratio (a/d) for all beam specimens is approximately 3. The 16-inch wide beams were proportioned so that the shear stress associated with yielding of the #3 stirrups, spaced at $d/4$, does not exceed $6\sqrt{f'_c}\, b.d$ [lb,in.], see Table 4.1 for details.

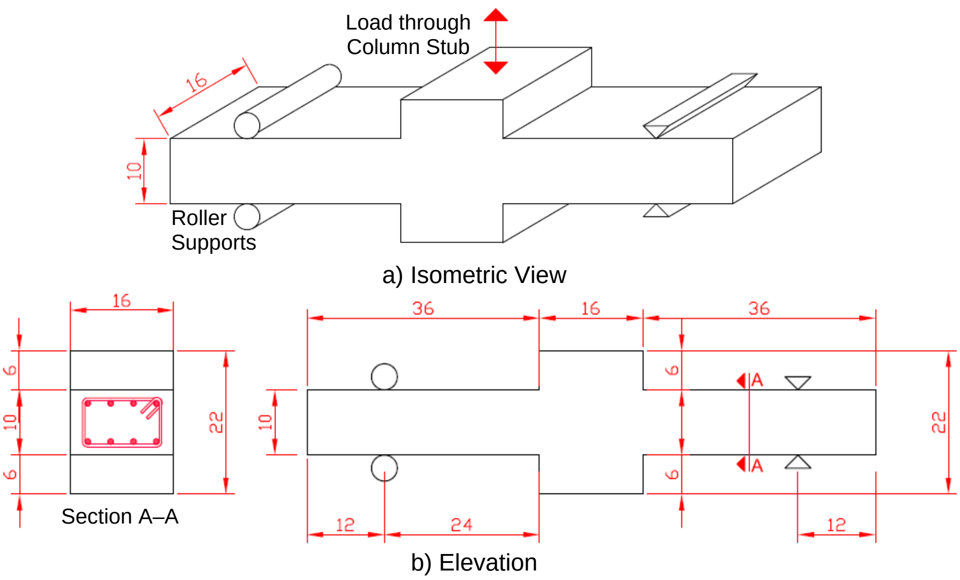

Figure 3 – Geometry of a Typical Beam Specimen (dimensions in inch).

A total of 8 beam specimens are planned for testing. In each sequence of testing, from specimen #1 to specimen #8, only one parameter is changed. For some specimens with fibers, the spacing is relaxed to $d/2$ to investigate the effects of fibers on confinement and shear. All specimens with fibers are HPFRC with 1.5% volume fraction of steel-hooked fibers (Dramix RC-80/30-BP) with a length of 1.2 in. (30 mm) and a diameter of 0.015 in. (0.38 mm), for a fiber aspect ratio of 80. These fibers are commercially available and manufactured by N.V. Bekaert S.A., Belgium.
In each specimen, two longitudinal bars are instrumented with seven strain gages (see Fig. 4). The strain gages for the longitudinal bars are YEFLA-5-5L rated for 10% strain and are supplied by Texas Measurements, Inc. (TMI), www.straingage.com. Transverse reinforcement within a distance equal to the member depth from the stub face are instrumented with strain gages type FLA-3-5L rated for 2% strain, also provided by TMI.
The applied load will be measured using load cells attached to the actuator. The deflection of each beam will be measured using linear variable displacement transducers (LVDT). The curvature of each beam will be measured using potentiometers on each side of the column stub attached to the top and bottom of each beam with a gage length of 6 in. The shear deformations at the location of the expected plastic hinges will be measured using diagonal potentiometers attached at the opposite faces of each beam. All measurements will be recorded using a digital data acquisition system. The location of LVDTs and potentiometers are shown in Fig. 5.

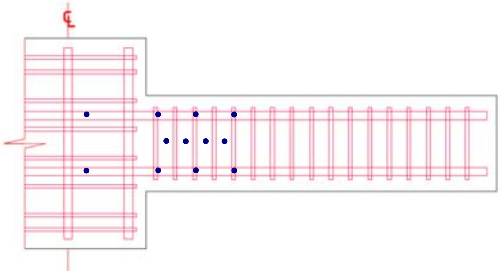

Figure 4 – Strain Gage Locations.

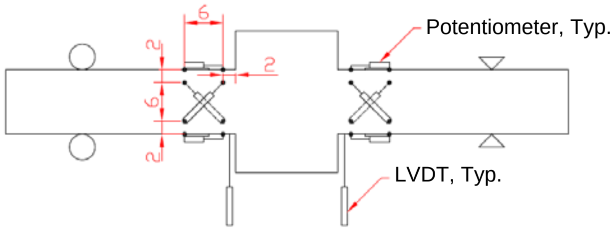

Figure 5 – Instrumentation Layout.

4.2. Column Tests

To evaluate the behavior of concrete columns reinforced with UHSS, a series of experiments will be conducted at Purdue University. A summary description of the specimens is shown in Table 4.2. The main variables are:

- Yield strength of longitudinal reinforcement, f_y = 60 and 120 ksi [410 and 830 MPa]
- Yield strength of transverse reinforcement, f_y = 60, 120, and 185 ksi [410, 830, and 1280 MPa]
- Steel fiber-reinforced concrete (V_f = 0% and 1.5%)
- Spacing of transverse reinforcement ($s = d/4$ and $d/2$)

The column specimens have different geometry from the beam specimens but use a similar test setup, see Fig. 6.

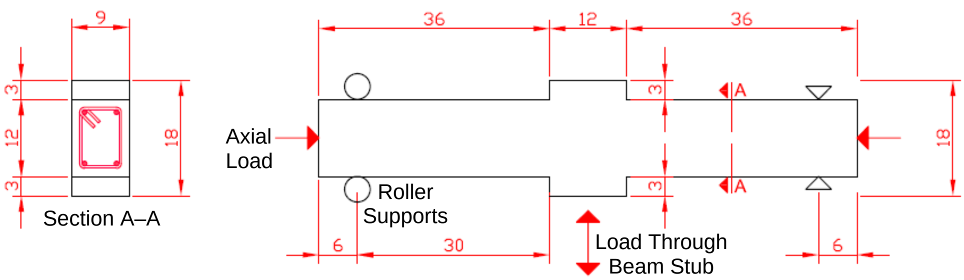

Figure 6 – Geometry of a Typical Column Specimen (dimensions in inch).

As indicated in Table 4.2, a total of 6 column specimens are part of the experimental program. As in the tests of beams, subsequent tests intend to introduce a variation in a single parameter. The target concrete strength is 6000 psi [41 Mpa] and longitudinal and transverse reinforcement are #6 or #7 and #3 bars, respectively, with nominal yield strengths as shown in Table 4.2. The axial force for all specimens is approximately $0.10\,f'_c.A_g$. Hoop spacings of $d/4$ and $d/2$ (but not more than six times the longitudinal bar diameter) are considered. Fiber-reinforced concrete is going to be made with steel-hooked fibers identical to those used in the beam specimens described in Table 4.1.

Table 4.2 Summary of Proposed Column Tests (Purdue University)

#	Specimen[a]	Longitudinal			Transverse (2#3)		M_{pr} [b]	V_{pr} [c]	V_s [d]	$V_{pr} - V_s \geq 0$
		Top	Bottom	f_y , ksi	Spacing, in	f_y , ksi	$b.d\sqrt{f'_c}$	$b.d\sqrt{f'_c}$	$b.d\sqrt{f'_c}$	$b.d\sqrt{f'_c}$
1	CC4-X	3#7	3#7	60	2.5	60	105	6.0	7.6	0.0
2	UC4-X	2#6	2#6	120	2.5	60	110	6.3	7.6	0.0
3	UC4-F	2#6	2#6	120	2.5	60	110	6.3	7.6	0.0
4	UU2-F	2#6	2#6	120	4.5	120	110	6.3	8.4	0.0
5	CU2-X	2#6	2#6	120	4.5	185	110	6.3	13.0	0.0
6	UU2-X	2#6	2#6	120	4.5	120	110	6.3	8.4	0.0

All column specimens are subjected to an axial load of 60 kip [267 kN] or ~ $0.10\,f'_c\,.\,A_g$.
See Table 4.1 for additional footnotes.

4.3. Loading Protocol

The specimens are going to be loaded through the central stub. The displacement of the stub will be controlled taking into account its rotation. It is known that the loading history affects the mechanical behavior and ultimate deformation capacity of reinforced concrete members[24]. The cyclic loading to be used in this research is the one recently suggested in FEMA 461[25]. The loading history contains repeated cycles of step-wise increasing deformation amplitudes. In order to satisfy the ATC/FEMA protocol, any amplitude should be repeated twice and a minimum number of 10 steps (i.e. 20 cycles) should be performed up to the target deformation. In each step, the amplitude is to be increased by 40%. Based on FEMA 461 guidelines, the loading history presented in Table 4.3 (illustrated in Fig. 7) will be adopted, where the 10th-step target deformation is taken as 3% drift. Drift is defined as the displacement of the central stub divided by the shear span ratio plus the rotation of the stub.

Table 4.3 Loading Protocol.

Step[a]	Drift (%)
1	0.15
2	0.20
3	0.30
4	0.40
5	0.60
6	0.80
7	1.00
8	1.50
9	2.00
10	3.00
11	4.00
12	5.00
13	7.00
14	8.00
15	9.00

[a] Two cycles of loading at each step

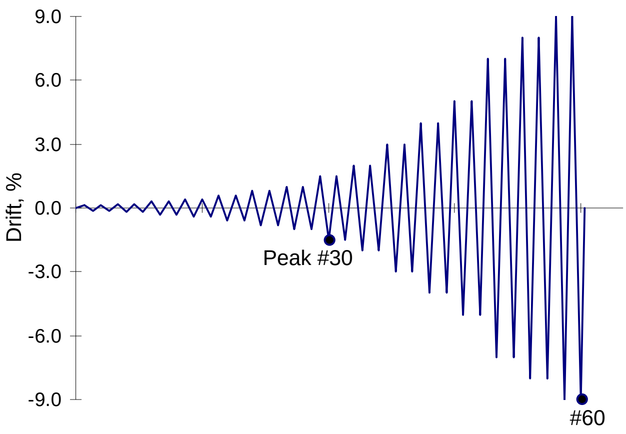

Figure 7 – Loading Protocol.

CONCLUDING REMARKS

A collaborative experimental program between Penn State University and Purdue University is underway to investigate the deformation capacity of reinforced concrete members reinforced with ultra high-strength steel reinforcement. The members are subjected to combined shear, moment, and axial load, applied through controlled increasing displacement reversals. Main variables of the experiments include: yield strength of main longitudinal reinforcement, 60, 100, and 120 ksi [410, 690, and 830 MPa]; yield strength of transverse reinforcement, 60, 120, and 185 ksi [410, 690, and 1280 MPa]; spacing of transverse reinforcement, $d/2$ and $d/4$; volume fraction of steel fibers, 0 and 1.5%; ratio of compression-to-tension longitudinal reinforcement, ρ'/ρ = 0.5 and 1.0; and type of loading, monotonic and cyclical.

Within the range of code-accepted limits on reinforcement ratios, shear stress levels, and length-to-depth ratios, it is expected that the deformation capacity of ultra high-strength steel reinforced concrete members is going to be increased by (1) reductions in spacing of transverse reinforcement; (2) increases in the ratio of compression-to-tension longitudinal reinforcement, and/or (3) addition of engineered fibers.

It is expected that this pioneering experimental program on the combined use of ductile nonprestressed ultra high-strength steel and fiber-reinforced concrete will promote the use of a new class of high-performance material in earthquake-resistant construction.

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