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PERFORMANCE OF PREFABRICATED HIGH STRENGTH WELDED WIRE GRIDS IN DUCTILE CONCRETE SHEAR WALL BOUNDARY ELEMENTS

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SUMMARY

The performance of a new proprietary concrete reinforcement product, BauMesh, when used as confinement reinforcement in ductile concrete shear walls is discussed.

Large material and labor savings are achieved when concrete shear walls are reinforced with these new high strength welded wire ladders positioned at close spacing (3″ o.c.) along the length of the longitudinal reinforcement to ensure a non-brittle ductile response to violent earthquake generated cyclic forces.

Design, quality control during manufacture, installation, and the observed performance of a 17-story San Francisco ductile shear wall building during the Loma Prieta earthquake are discussed.

1. INTRODUCTION

Reinforced concrete elements subjected to compression in plastic hinge regions must be confined with transverse reinforcement to ensure stable hysteretic behavior under severe inelastic deformations. The stable inelastic behavior of a structure will considerably improve the structure's energy dissipation capacity which in turn will improve the overall behavior of the structure in the event of an extreme excitation. The ability of a structure to displace beyond its elastic limit is commonly referred to as displacement ductility. The ratio of maximum displacement (δ_{max}) to yield displacement (δ_y) is referred to as the displacement ductility ratio ($\mu_\delta = \delta_{max}/\delta_y$). To develop displacement ductility, the subassemblages of a structure must be able to develop rotation ductility ratios ($\mu_\theta = \theta_{max}/\theta_y$) in the plastic hinge regions. The development of adequate rotation ductility ratios in plastic hinge regions is dependent upon the development of adequate curvature ductility ratios ($\mu_\phi = \phi_{max}/\phi_y$) in the cross-sections along the lengths of the plastic hinges and the development of sufficient length of the plastic hinge (length of the critical region where yielding occurs). To develop curvature ductility ratios in the section of a member, it is necessary to develop adequate strain ductility ratios ($\mu_\varepsilon = \varepsilon_{max}/\varepsilon_y$) in the extreme fibers of the cross-section. Therefore, it can be said that to develop μ_δ in a structure, it is necessary to develop μ_θ in the plastic hinge regions, average μ_ϕ in the cross-sections of members within the plastic hinge length, and ultimately μ_ε in the extreme fibers of the sections within the plastic hinge regions. Although the displacement, rotation, average curvature and strain ductility ratios are related, they are not equal.

Plain reinforced concrete does not possess adequate strain ductility to attain the maximum compressive strains required in these plastic hinge regions, which may be well beyond the use for limit strains (ε_{cu}) of 0·003 to 0·004 characteristic of plain reinforced concrete. The use of

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transverse reinforcement in reinforced concrete compressive elements can increase the attainable concrete compressive strains by confining the concrete core area enclosed by the transverse reinforcement. In frame–wall reinforced concrete structures, the columns, boundary elements of shear walls, and girders must have adequate transverse reinforcement such that required structure displacement ductility ratios can be realized if a design approach which explicitly or implicitly is based upon the development of inelastic behavior in the structure is utilized.

The functions of transverse reinforcement in reinforced concrete construction are:

(i) to confine the concrete;
(ii) to resist shear;
(iii) to prevent or delay reinforcement buckling; and
(iv) to basket the concrete in the core area, improving the bond of bars anchored in this area.

The design philosophy of transverse reinforcement reflected in the 1991 Uniform Building Code[1] is to ensure that adequate transverse reinforcement exists such that the nominal strength of the section prior to spalling of the unconfined concrete cover is less than or equal to the strength of the section with the cover spalled when the concrete core strength is computed based upon confined concrete properties. The spacing limits for transverse steel are intended so that a shear crack will cross at least one tie, hence precluding the opening of wide shear cracks when the shear carried by the transverse hoops (V_s) is greater than or equal to twice the nominal shear capacity of the concrete shear area (V_c). The provisions for transverse reinforcement are intended to resist the effects of shear and to ensure that the ties are able to prevent longitudinal reinforcement buckling.

The use of high strength welded wire grids (WWG) as transverse reinforcement offers the advantages of a more efficient use of material and especially ease of fabrication and construction. Analytical relationships for the stress–strain relationship of confined concrete[2,3,4] implicitly suggest that the use of higher strength transverse reinforcement will increase the effectiveness of the confinement.

Transverse steel incorporated in reinforced concrete construction is predominantly deformed bars formed into closed hoops and supplementary cross tie bars. The difficulty in consistently forming a uniform closed hoop shape and placing the reinforcement in members renders the use of prefabricated ties fabricated to uniform dimensions a desirable alternative to traditional deformed ties.

Baumann Engineering has proposed a lateral tie fabricated from high strength wire cut and automatically welded into rectangular grids designed to accommodate various wall thicknesses.[5] These grids, a proprietary product of the BauMesh Company in Newport Beach, California, U.S.A., will be referred to herein as BauMesh.

2. BAUMESH

Over many years, the structural engineering community has been working to develop the safer use of reinforced concrete structures in regions of high earthquake risk throughout the world. In the forefront of these efforts has been the need to find the right blend of steel reinforcement and concrete such that the structural qualities of each are combined to create strong and secure walls, columns and beams while at the same time, allowing for a structure that is ductile enough to remain intact under the most extreme destructive seismic forces.

Until recently almost without exception, individual re-bar hooked hoop ties and stirrups have been used to tie the major reinforcing steel elements together to resist the bursting forces that

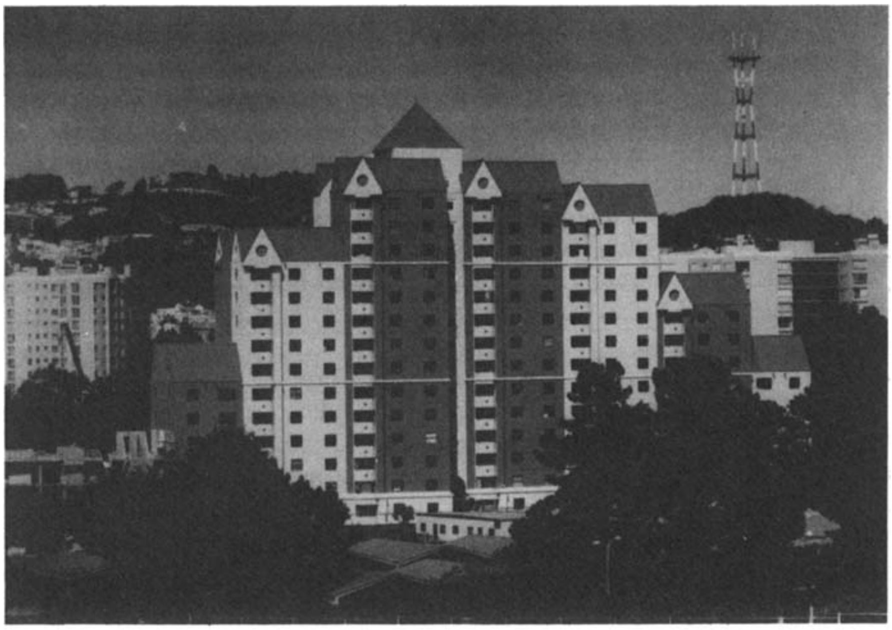


Figure 1. Residence dormitory on campus of San Francisco State University

are caused by large seismic forces. These ties have proven to be costly to fabricate, time consuming to install and sometimes inadequate in fulfilling their designated purpose of assuring a more ductile response by the structure.

Recently, a high strength welded wire ladder device, known as BauMesh, has been developed and used in place of re-bar ties. Although the concept of a welded 'wrap around' confinement fabric has been used for many years, it was not until 1987 that an engineering and research firm in Newport Beach, California, invented and developed a new welded confinement reinforcement device to the point of actually having it tested, fabricated, specified and used in concrete structures in seismic zones.

Under the direction of the product development engineer and the inventor of BauMesh, Hanns U. Baumann, President of the Baumann Research and Development Corporation, the new BauMesh product was approved and used on two projects. The first was a 17-story high rise dormitory on the campus of San Francisco State University (SFSU) and the second a 60 unit 4-story apartment complex in Long Beach, California.

Although the two jobs were quite different in size and scope, the responses to the use of the product were very favorable. On both jobs, BauMesh was found to be faster to install and less costly to work with.

Figure 1 shows a photograph of the Residence Apartment building at San Francisco State University. Load bearing shear walls, 7″ thick, at 12′6″ o.c. support $5\frac{1}{2}$″ thick one way slabs and provide the building's lateral force resisting system in the short direction. In the long direction the corresponding lateral force resisting system is composed of 12″ thick ductile shear walls along the central hallway.

The present day 'tunnel form' method of construction prohibits the use of barbell type shear walls. However, its speed and economy still make it a very attractive way to build shear wall type structures for dormitory, hotel and apartment buildings.

Tunnel form manufacturers are now working on form systems to construct barbell type shear walls for taller buildings in seismic zones.

The problem during the design of the SFSU building was to achieve reliable ductility in 17-story high shear walls only 7″ thick.

Research has shown that building in greater ductility is very desirable, because the structures will then 'attract' significantly less earthquake force.[6,7]

The lack of consistent dimensions in re-bar confinement hoops as well as the costly labor to install the many individual pieces has prompted a search for a better way to achieve reliable ductility in reinforced concrete.

In place of hooked re-bar hoops, a new proprietary product called BauMesh has recently been introduced. For shear walls the BauMesh has a ladder configuration in strips, tees and L-plan. Instead of the $\pm\frac{1}{2}''$ dimensional tolerance of re-bar hoops, BauMesh is manufactured of high strength wire (up to 80 ksi) to $\pm\frac{1}{16}''$ dimensional accuracy.

Cages assembled with large longitudinal re-bar confined by BauMesh perform consistently well because of the uniform confinement the full length of the boundary element. During erection the boundary cages stand very erect and do not corkscrew, which saves very significant bracing costs and crane time.

2.1. Final design of 17-story structure

Shown in Figure 2 is a typical 7″ thick transverse load bearing ductile shear wall. Important to the economy of the system is locating the resisting reinforcement as far as possible from the center of the wall length. The BauMesh ladders are designed to allow this concentration of re-bar and still leave a vertical passageway for wet concrete and the vibrator every ten inches along the wall's length.

2.2. BauMesh installation

For the construction of the SFSU building, compact bundles of BauMesh ladders were delivered to the site. Using conventional horses, the two upper-most re-bar are charged through the bundle. The BauMesh now suspended from these bars is spread out and positioned at the standard 3″ spacing. Subsequent longitudinal bars are charged so that each cell holds four #11 re-bar at 10″ on center.

The boundary cages, four floors high, are made and lifted in one piece which saves significant assembly and erection crew labor and crane time.

The inherent rigidity of the many welded joints, and the dimensional accuracy, make possible rapid installation of BauMesh cages in thin wall forms. Newly developed assembly jigs allow further automation resulting in significantly increased productivity (250%) of the workers.

2.3. Structural response to Loma Prieta earthquake

The Loma Prieta earthquake struck the SFSU project when it was completed up to and including in part the 14th floor, so in essence the concrete structure was 90% completed.

The construction workers felt very significant movement at the 14th floor where they were pouring concrete.

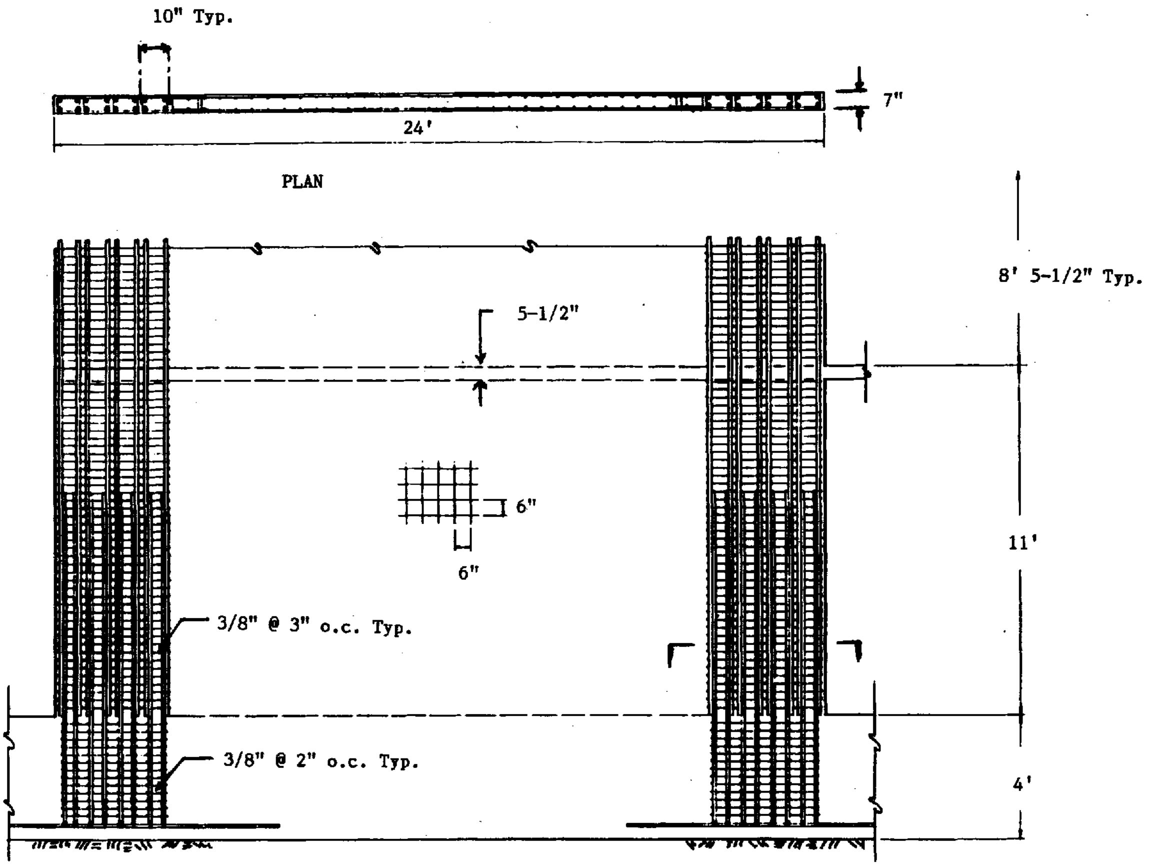


Figure 2. Transverse shear wall reinforcement

Upon visual inspection, no evidence of distress was found. In fact the hairline cracks normally observed in reinforced concrete subjected to this level of ground motion were not found.

2.4. Statement of the problem

The test program described herein was designed to study the mechanical behavior of six shear wall boundary elements which employed BauMesh as transverse reinforcement. The specimens measured 5·5″ × 18″ (plan) in the test region and were 4′8″ tall.

The specimens were designed to model boundary elements of uniform width shear walls of the dormitory structures under construction at San Francisco State University using tunnelling construction techniques. A typical section of three sizes of wall used in the structure is presented in Figure 3. The project is located on the east side of Lake Merced in San Francisco, approximately 3 miles from the San Andreas Fault.

The seismic design of the dormitory structure was based upon an inelastic design spectra constructed using methods developed by Newmark and Hall.[8] To construct the design spectra, a displacement ductility ratio of five was used by the structural engineers. The curvature ductility ratio (μ_ϕ) necessary to attain a given displacement ductility ratio ($\mu_\delta = 5$) can be approximated

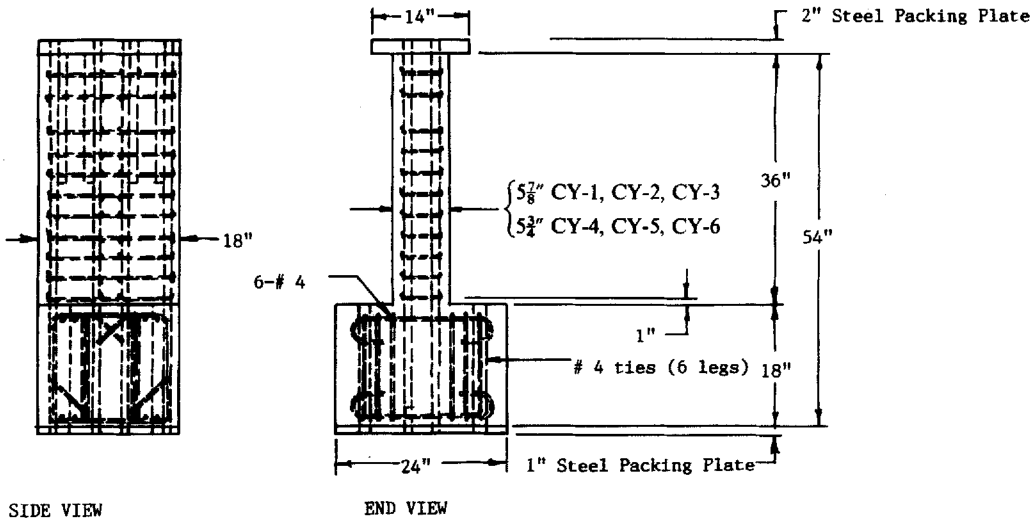


Figure 3. Specimens CY-1–CY-6 schematic

using Figure 4, which uses an equation developed by Paulay and Uzumeri.[9] The range of wall lengths on the project is 20′0″ to 25′6″ and the range of wall heights on the project is 122′ to 147′. Therefore, the envelope of values for aspect ratios of these walls is 4·8 to 7·4. It can be shown that the curvature ductility ratios that must be supplied by the wall sections can be up to approximately $\mu_\phi = 14$. Moment curvature analyses were conducted for a variety of wall cross sections which had a variation of boundary element sizes and reinforcement. Using typical moment curvature relationships for three example wall cross-sections of the SFSU project, it was determined that the maximum strains that must be obtained by the boundary elements were $\varepsilon_{max} \approx 0{\cdot}01$ and $\varepsilon_{max} \approx 0{\cdot}06$ in compression and tension, respectively. The major objective of this paper is to describe the mechanical behavior of shear wall boundary elements incorporating BauMesh for transverse reinforcement under severe cycles of axial load reversals.

2.5. Description of test specimens

Six specimens were constructed for the test program. The specimens were designed as full-scale replicas (in width dimension and longitudinal reinforcement size) of portions of boundary elements found in shear walls typical of the SFSU dormitory project. The specimens were designated as CY-1, CY-2, CY-3, CY-4, CY-5 and CY-6.

The ends of specimens were designed so as to create a critical region of the specimen which enabled a more detailed study of the region and eliminated problems associated with end region failure due to possible stress concentrations in the end regions of the specimens. In the critical region, all six specimens incorporated the minimum cover that it was possible to construct in order to have more control in the post-spalling phase of the testing.

The critical section dimensions of specimens CY-1, CY-2 and CY-3 were $5\frac{7}{8}'' \times 18''$, while for specimens CY-4, CY-5 and CY-6 they were $5\frac{3}{4}'' \times 18''$. Detailed sketches of these specimens are presented in Figure 3. A summary of the specimens and their design properties is presented in Table I.

The longitudinal reinforcement ratio ($\rho = A_s/A_g$) of the first three specimens was 6·0% while that of the second three specimens was 6·1%. This amount of longitudinal reinforcement is equal

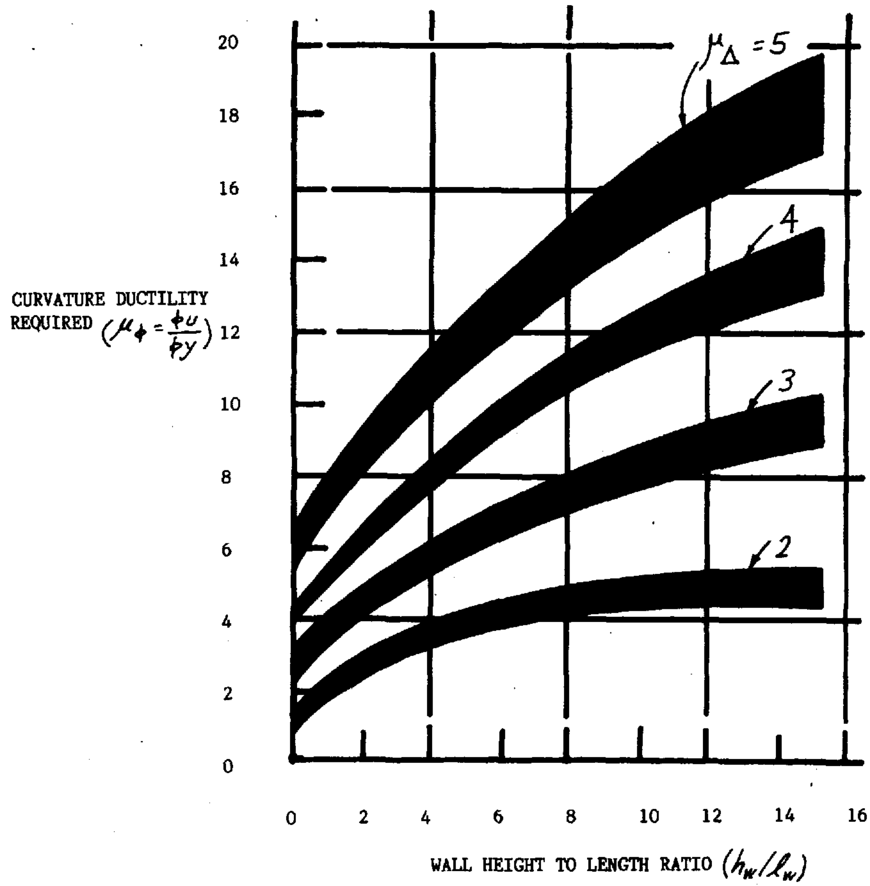


Figure 4. Variation of μ_ϕ at the base of cantilever walls with the aspect ratio and imposed μ_Δ demand

Table I. Specimen design summary

Specimen	Gross dimensions	Long. reinf. Bars	ρ_g	Transverse reinforcement Type*	Size	Spacing	ρ_s
CY-1	$5\frac{7}{8}'' \times 18''$	8–#8	0·060	BM	$\frac{3}{8}''$	3″	0·020
CY-2	$5\frac{7}{8}'' \times 18''$	8–#8	0·060	BM	$\frac{3}{8}''$	3″	0·020
CY-3	$5\frac{7}{8}'' \times 18''$	8–#8	0·060	BM	$\frac{3}{8}''$	6″	0·010
CY-4	$5\frac{3}{4}'' \times 18''$	8–#8	0·061	BM	$\frac{1}{4}''$	3″	0·0096
CY-5	$5\frac{3}{4}'' \times 18''$	8–#8	0·061	BM	$\frac{1}{4}''$	6″	0·0048
CY-6	$5\frac{3}{4}'' \times 18''$	8–#8	0·061	BM	$\frac{1}{4}''$	6″	0·0048

* BM = BAUMESH module

to or exceeds the maximum steel ratio of 6% permissible by the UBC[1] for compression members of frame members of structures located in regions of high seismic risk. However, the UBC does not explicitly limit the longitudinal reinforcement ratio of the boundary elements of shear walls. For this steel ratio, the placement of concrete during construction usually represents an important problem. However, this is not the case with BauMesh's 'vibrator cell', each 10″ o.c.

The volumetric ratio of the transverse reinforcement (ρ_s), the volume ratio of confining steel to confined concrete within a hoop spacing, ranged from 2·0% for specimens CY-1 and CY-2 to 0·48% for specimens CY-5 and CY-6. It is important to mention that specimens CY-3, CY-4, CY-5 and CY-6 did not comply with the 1991 UBC specifications and were incorporated to study confinement near but outside of the plastic hinge regions.

2.6. Fabrication and casting of the specimens

To obtain a uniform distribution of stresses in the instrumented region of the specimen and to provide an adequate length for the development of the failure mechanism, the minimum width-to-length ratio of three specimens was used.

In order to mitigate end region stress concentrations and possible end region failure, the spacing of transverse reinforcement in the end regions of the specimens was reduced to two inches in each specimen.

In the upper part of the specimens an external clamping device was incorporated to provide additional confinement. In the lower end the additional confinement was provided by an enlarged area as shown in Figure 3. To increase the strength, both in compression and tension of the ends of the specimens additional longitudinal reinforcement was placed. Steel end-plates were cast on the ends of these specimens in order to provide both a good bearing surface and an anchoring surface for applying tension to the specimens.

The sequence of construction was (1) to build the reinforcement cage, (2) to place and secure the cage and instrumentation rods in the formwork, and (3) to place the concrete. The formwork was constructed of treated lumber. The concrete was placed in two well vibrated lifts.

The specimens were covered with moist burlap and polyethylene covers for a period of one week prior to stripping off the formwork.

To obtain a homogeneous concrete strength, all of the specimens were cast simultaneously. In addition to the specimens, 6″ × 12″ cylinders were cast for control specimens.

The concrete employed for specimens CY-1 through CY-6 was of mix design identical with that used in the SFSU dormitory project. A summary of the mix design is presented in Table II.

Table II. Concrete mix design

Material	Amount for one cubic yard batch
Type II Portland cement	583 lb
Water	292 lb
½″ × #8 gravel	1650 lb
Coarse sand	1119 lb
Fine sand	363 lb
WRDA 79	29 oz
WRDA 19	90 oz

Table III. #8 longitudinal reinforcement coupon test data: specimens CY-1–CY-6

	Strength (ksi)		Strain (in/in)			Modulus	
Bar	Yield	Max	ε_y	ε_{sh}	ε_{max}	Elastic	Strain hard.
A	74·0	115·0	0·0026	0·0010	0·18	28 460	1300
B	75·5	116·5	0·0028	0·0009	0·17	27 800	1225

The design strength of the concrete was 4000 psi.

The stress–strain relationships indicate a maximum strength of 5200 psi, a modulus of elasticity (E_c) of 3850 ksi and strain corresponding to a maximum stress (ε_0) of 0·0023. These values are in general agreement with what would be expected of concrete with a water-to-cement ratio similar to that used in the mix design.

Coupon tension tests for two #8 longitudinal bars used in the experiments were conducted. The longitudinal reinforcement bars were ASTM A615 grade 60 steel. These tests were conducted under load control in a Baldwin 120 kip C/T machine located in the Structures Laboratory in Davis Hall at the University of California.

The coupons were milled in a central region in order to create a known point of failure and an acceptable surface on which to attach the extensometer used to measure strains.

The design properties obtained in these tests are presented in Table III. The yield strength (f_y) of the coupons was 74·0 ksi and 75·5 ksi, based on the yield plateau, and was very clearly defined for both coupons. The ultimate strength (f_u) of the bars was 115 ksi and 116·5 ksi. The modulus of elasticity of the coupons was 28 460 ksi and 27 800 ksi, whch correlates reasonably well with normal ranges for mild reinforcing steel (29 000 ksi–30 000).

Coupon tension tests for the BauMesh wire used in the experiments were conducted. These tests were conducted under load control on a Baldwin 120 kip C/T machine located in the Structure Laboratory in Davis Hall at the University of California.

The coupons were milled in a central region in order to create a known point of failure and an acceptable surface on which to attach the extensometer used to measure strains.

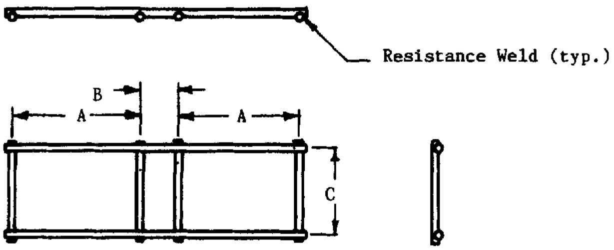
The design properties obtained in these tests are presented in Table IV.

The $\frac{3}{8}''$ wire, on the basis of the average of three coupon tests, had a yield strength (f_y) using the 0·2% offset method of 78·0 ksi, an ultimate strength (f_u) of 85·5 ksi and a modulus of elasticity of 30 300 ksi. The $\frac{1}{4}''$ wire, on the basis of the average of three coupon tests, had a yield strength (f_y) based on the 0·2% offset method of 81·5 ksi, an ultimate strength (f_u) of 93 ksi and a modulus of elasticity of 29 480 ksi. Neither the $\frac{3}{8}''$ wire or the $\frac{1}{4}''$ wire displayed a defined yield point and fracture strains were in the range of 6% to 9%. The stress–strain curves obtained, the yield and

Table IV. Transverse reinforcement coupon test data: specimens CY-1–CY-6

	Strength (ksi)		Strain (in/in)			Modules (ksi)	
Bar/wire size	Yield	Max	ε_y	ε_{sh}	ε_{max}	Elastic	Strain hard.
$\frac{1}{4}''$ dia.	81·5*	93·0	0·0048*	—	0·067	29 480	—
$\frac{3}{8}''$ dia.	78·0*	85·5	0·0045*	—	0·070	30 300	—

* Based upon 0·2% offset method



Wire Dia.	A	B	C
1/4"	7-5/8	2-1/4"	5-1/4"
3/8"	7-1/2"	2-1/4"	5-1/8"

Figure 5. BAUMESH schematic

ultimate strengths measured, and the moduli of elasticity established for these wires are characteristic of cold drawn steel wires.

The wire used for the BauMesh was obtained from cold drawn wire with a minimum specified yield strength corresponding to the 0·2% offset strain method of 75 ksi. The modules of wire were formed at Meadow Steel Products in Garden Grove, California, by resistance welding the wires into patterns as presented in Figure 5.

2.7. Experimental set-up and testing procedure

Displacement transducers were used to measure deformations within each specimen, and load cells recorded the imposed axial forces.

To measure the axial deformation in the critical region of the specimens, $7\frac{1}{2}''$ long rods having diameters of $\frac{3}{8}''$ were placed perpendicular to the longitudinal axis of the specimen. The rods were therefore embedded in the concrete across the entire section, with the exception of the region through the cover where the rods were isolated from the concrete by placing a piece of rubber hose which was removed after the concrete had cured. This ensured that the relative movement of the rods would not be affected by the spalling of the concrete cover. Vertical direct current displacement transducers (DCDTs) were attached to these rods and electroncially measured the displacements between the embedded rods. The average strains were then determined by dividing the average of the deformations measured by the DCDTs on opposite faces of the specimen. It should be noted that within the gauge lengths of the DCDTs strains may well have been higher than the average strains measured over the complete gauge lengths. The load in the specimens was measured by load cells mounted on the two loading actuators of the test frame. Additionally properly calibrated strain gauges were mounted on the longitudinal and transverse reinforcement to determine their load–strain histories.

The data acquisition system was based on Pacific Signal Conditioners and a digital LSI-11 multiplexer. The transducer output was passed through Pacific Signal Conditioners; these conditioners provide the excitation voltage to the transducers, amplify and then filter the analog output to 100 Hz. The digital multiplexer scans the signal conditioners and sequentially reads each channel at a maximum burst rate of 80 Hz. The maximum scanning rate (i.e. the number of times per second each channel is sampled) is controlled by the LSI-11 software. The analog output from the multiplexer is then passed through a Preston A/D converter, which converts the signal to a digital form for storage on the LSI-11 hard disk.

The specimens were tested in a compression test frame situated in the Earthquake Simulator Laboratory of the University of California at Berkeley.

The compression test frame, originally designed to test the hysteretic behaviour of rubber bearings, can accommodate specimens up to 60″ in height and 22″ in cross-section. The compression test frame consists of two vertical 1000 kip actuators attached to two WF 36X300 beams, which are in turn attached to a load application girder, constructed of two WF 24X162 sections and cover plates. The frame is mounted to a steel box section constructed of 2″ steel plate. The vertical actuators can attain maximum displacements of 6″. The maximum compression force that each actuator can deliver is 1000 kips. Additionally, the test frame was modified in order to be able to apply tensions to the specimen. The maximum tension force capacity of the test frame is 800 kip. For the experiments, the vertical actuators were run under displacement control so that the descending branch of the load–deformation curve for the specimens could be reliably captured. Sketches of the test frame with a typical specimen in position for testing are presented in Figures 6 and 7.

To ensure a good contact surface between the test machine and the specimens, the specimen ends were capped in place with Hydrostone. Hydrostone is a gypsum cement that cures and attains strength in short periods of time.

The specimens were tested cyclicly in compression and tension, at a low rate of monotonic loading (of the order of 30 $\mu\varepsilon$/s), to ensure that strain rate effects were not introduced into the experiments. The global load–strain behavior of each specimen was monitored with a X–Y–Y' recorder. Until the cover was completely spalled, the behavior was closely monitored and the test was interrupted if a sudden decrease in load was observed. This procedure was followed on all specimens to prevent any sudden increase in strain rate in the specimens due to the release of strain energy in the test machine when the strength of specimens was decreased due to the failure of the concrete cover. The problems associated with this sudden increase in strain rate were minimized by casting the specimens with as little cover as feasible.

The compression load was applied until a local strain of approximately 0·006 was attained; at this point the load was reversed and tension was applied until a local strain of approximately 0·06 was reached. The number of cycles applied to each case varied depending on the resistance of each specimen.

The test was considered complete when the resistance of the specimen decreased suddenly and the specimen exhibited signs of instability.

2.8. *Experimental results*

2.8.1. Specimen CY-1. Specimen CY-1 was cycled in compression several times (without reversing the load) until a global deformation of approximately 0·6% was achieved. At this point the load was reversed, applying tension to the specimen until a local strain of nearly 6% was attained. When the load was reversed in the second compression cycle, the uniformly distributed cracks that resulted from the tensile cycle were closed successfully. The point at which the cracks are closed is clearly evident in the observed load–strain relationship, which is shown in Figure 8. After closing the cracks, an important gain in stiffness was attained. In the second tension cycle the Bausching effect was clearly visible. Again, tension was applied until a local strain of nearly 6% was attained. When the load was reversed to apply compression for the third time, the cracks in the specimen were not completely closed when global buckling (lateral instability) was observed.

The failure was a global buckling mode of failure and the concrete core was still in relatively good shape at this point. The maximum strength attained by this specimen was 1040 kip in

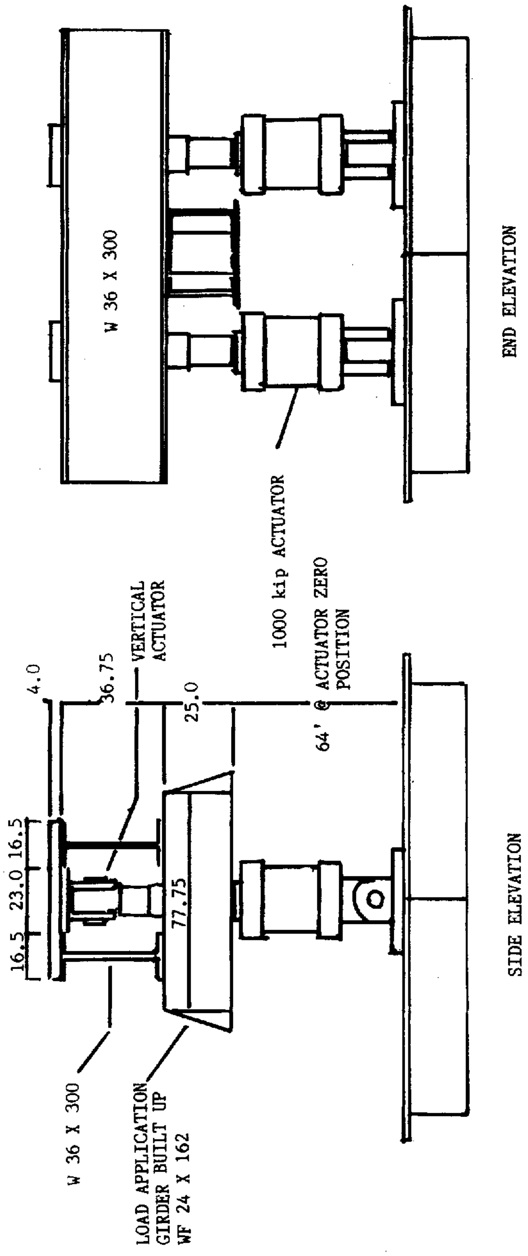


Figure 6. Compression test frame schematic

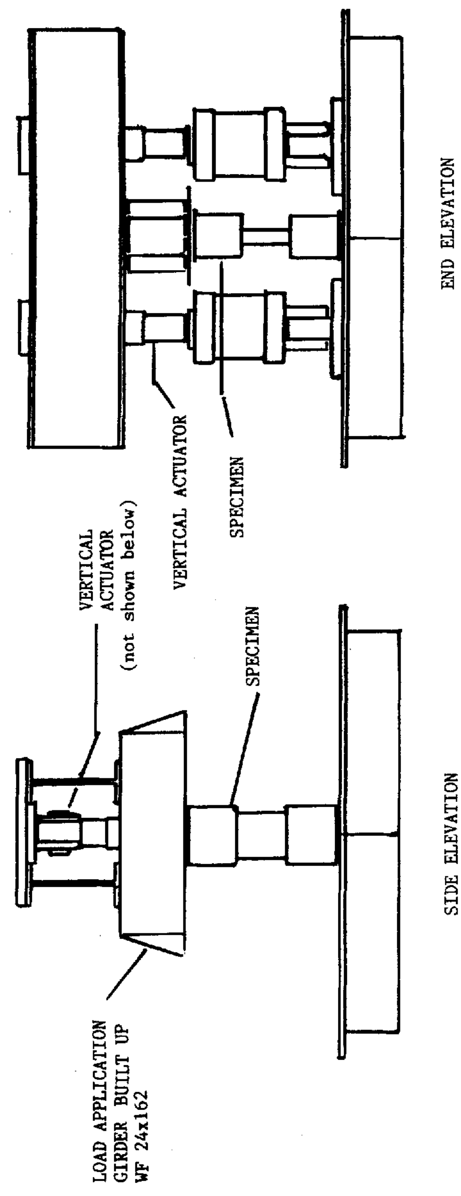


Figure 7. Schematic of typical specimen in test frame

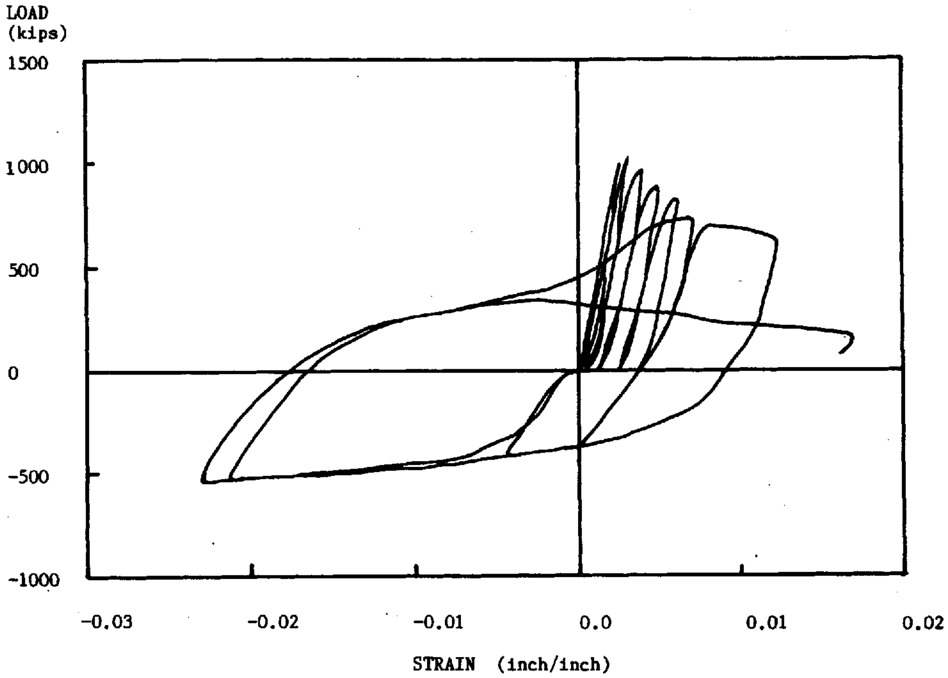


Figure 8. Observed load–strain behavior of specimen CY-1

compression and 530 kip in tension. After two-and-a-half cycles the maximum compression load that the specimen was able to carry was 360 kip. In the last compression cycle, the failure of one BauMesh module at a welding point was observed.

2.8.2. Specimen CY-2. Even though specimens CY-1 and CY-2 had exactly the same design the behavior of specimen CY-2 was somewhat different. Initially the specimen was subjected to smaller global deformations (in compression) than those imposed on CY-1. In the first cycle, the maximum compression strength was 1 105 kip. In tension, the specimen had a strength of 520 kip, which was observed to be the same in all three of the cycles that it was able to sustain. During the first two cycles in which the load was reversed from tension into compression, the cracks were able to be closed completely and a clear gain in stiffness was observed in the hysteretic behaviour. After three complete cycles, when the load was reversed to apply compression again a global instability failure was observed, occurring soon after closure of the cracks. The maximum average concrete compression strain was 0·02 and the steel strain in tension was 0·06.

Figure 9 shows the observed hysteretic behavior for this specimen. This load–strain relationship corresponds to global deformations that are close to those of the total length of the critical region of the specimen. The corresponding local deformations are shown in Figure 10. By comparing these two figures it is evident that local and global deformations are quite different, indicating that the deformation tends to concentrate in a certain region of the specimen. A global buckling mode of failure was clearly visible in the test.

As in the case of specimen CY-1, the concrete core held up relatively well to the failure. Failure

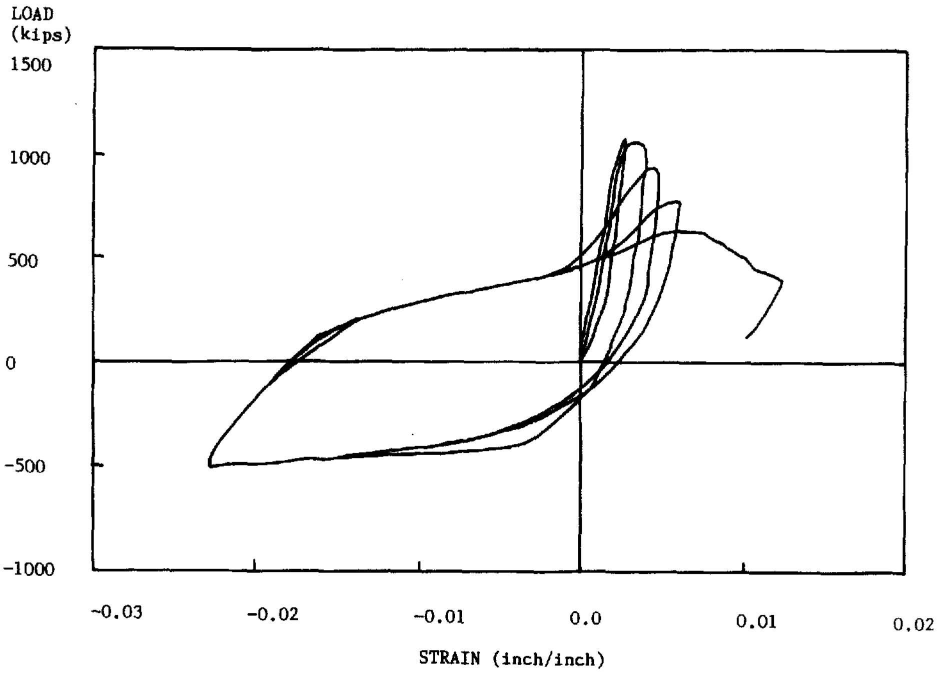


Figure 9. Observed load–global strain behavior of specimen CY-2

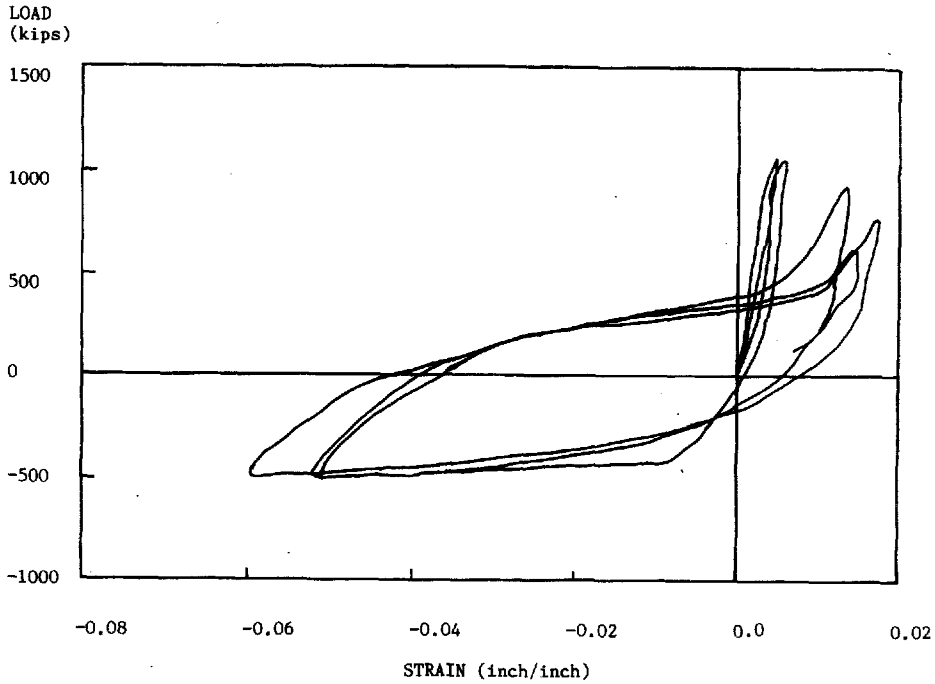


Figure 10. Observed load–local strain behavior of specimen CY-2

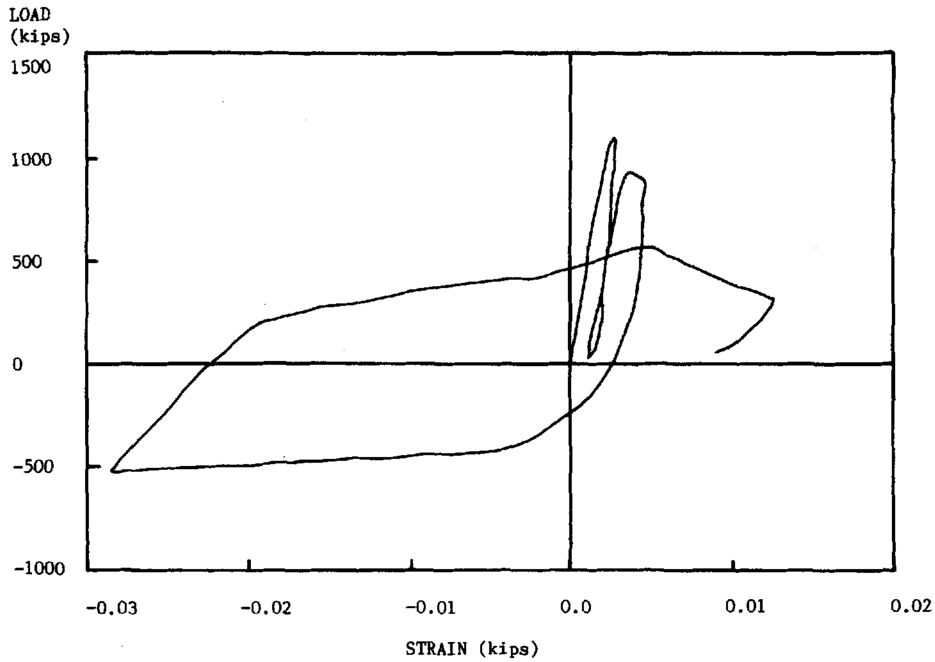


Figure 11. Observed load–global strain behavior of specimen CY-3

of two BauMesh prefabricated grids was observed in the last half cycle. One occurred at a corner welding point whereas the other occurred in the wire in the heat affected region near the welding of one of the cross ties. Before failure, this specimen was able to carry 660 kip in compression.

2.8.3. Specimen CY-3. Specimen CY-3 exhibited stable behavior in the first cycle, but it failed due to global instability (buckling) during the second compression cycle soon after closure of the cracks. Figure 11 presents the observed load–strain relationship for this specimen. This figure corresponds to global deformations close to those of the total length of the critical region of the specimen. The corresponding local deformations are shown in Figure 12. In this case, the maximum local strain in tension was close to 7‰. From this figure it can be seen that failure occurred when the increase in stiffness due to closure of the previously opened cracks was starting to occur.

The maximum compression strength observed was 1 120 kip in compression and 520 kip in tension. Before buckling started to occur, during the second cycle, a strength of 570 kip in compression was attained. The failure mode was global buckling and the concrete core did not have enough confinement, which resulted in premature failure of the core.

2.8.4. Specimen CY-4.. Specimen CY-4 had a similar behavior to that of specimen CY-3. This specimen also failed during the second compression cycle. Despite having similar transverse reinforcement, CY-4 failed sooner than CY-3.

The main factor that contributed to the earlier failure of this specimen with respect to CY-3 was the poor performance of the $\frac{1}{4}''$ BauMesh hoops spaced at 3″ to confine the concrete core. In the first compression cycle, the failure of one hoop was observed. After having reversed the

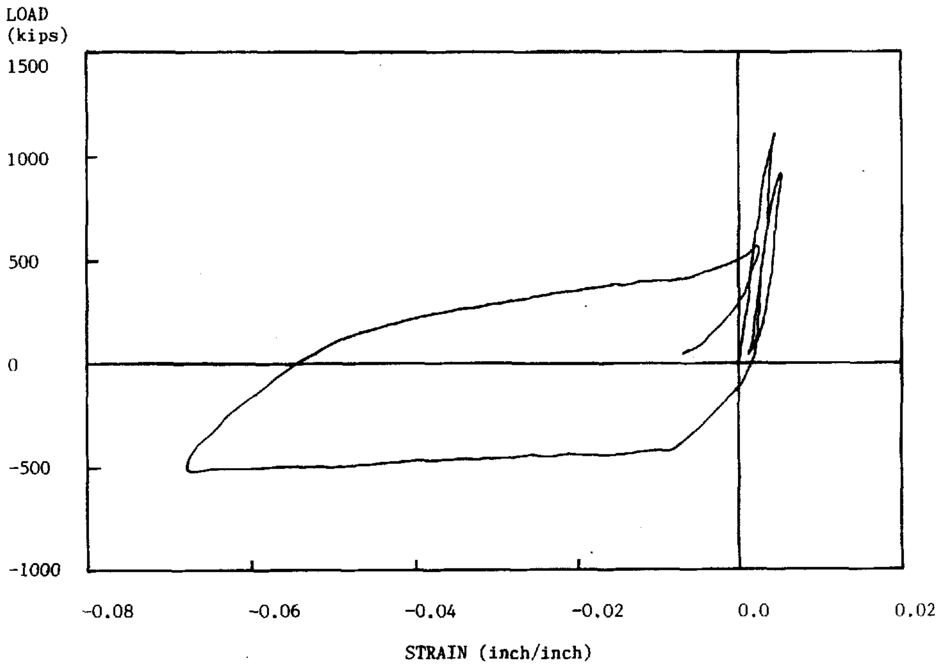


Figure 12. Observed load–local strain behavior of specimen CY-3

load from tension into compression, the BauMesh hoops failed almost simultaneously. In all cases the failure occurred at the welding points.

The maximum strength observed in this specimen was 1 020 kip in compression and 530 kip in tension. In the second compression cycle, a maximum load of only 195 kip was observed.

2.8.5. Specimen CY-5. Specimen CY-5 failed in a brittle fashion. The observed mode of failure was local buckling of the longitudinal reinforcement due to inadequate lateral restraint provided by the $\frac{1}{4}''$ BauMesh hoops spaced at 6″.

The specimen failed in the first compression cycle without being able to apply tension to the specimen.

The maximum strength measured in this case was 998 kip. The concrete of this specimen failed in a similar way to that of completely unconfined concrete.

2.8.6. Specimen CY-6. Specimen CY-6 failed in an extremely brittle fashion. Similarly to CY-5, the observed mode of failure was local buckling of the longitudinal reinforcement due to inadequate lateral restraint. The behavior of this specimen was almost that of completely unconfined concrete. The maximum compressive strength of this specimen was 1 010 kip. The premature failure of transverse reinforcement produced brittle failure of the concrete core almost as if it was plain concrete, and consequently produced local buckling of the longitudinal reinforcement. The test loading was much greater than the anticipated actual loads outside the plastic hinge region.

2.9. Evaluation of the experimental results

The maximum compressive strain that could be estimated to be developed in the extreme compression fibers of boundary elements of typical shear walls of the SFSU dormitories was approximately $\varepsilon_c = 0{\cdot}01$. The corresponding tensile strains were approximately $\varepsilon = 0{\cdot}06$. These strains were based upon the use of a displacement ductility ratio (μ_δ) of five for the structural system of the SFSU dormitories. Therefore, the behavior of the specimens up to strains of $\varepsilon_c = 0{\cdot}01$ in compression and of $\varepsilon = 0{\cdot}06$ in tension is of particular interest for this study.

The experimental program was designed to evaluate the behavior of a variety of BauMesh spacings and diameters, using specimens which were representative of typical boundary elements found in the shear walls of the SFSU dormitories.

Specimens CY-1, CY-2, CY-3 and CY-4 failed due to global buckling (lateral instability). The primary reason for such behavior is that when the reinforced concrete specimen was subjected to cyclic loading during the tensile part of the cycle, large cracks opened in the boundary element. During the subsequent reversal of wall displacements and hence unloading, the tensile stresses in the longitudinal bars reduced to zero. The application of a compression force applied to the boundary element at this point produced compression stresses in the longitudinal bars. Unless the cracks close, the entire internal compression within the section must be resisted by the vertical reinforcement. Owing to the Bauschinger effect, the modulus of elasticity for steel is reduced at this stage. Out-of-plane buckling may commence when the tangent modulus of steel reaches a critically small value, while residual cracks in the concrete are still open. This is what was observed in the case of specimen CY-4 where instability occurred before the residual cracks closed.

If horizontal cracks across the boundary element of the wall close before such a critical stage is reached, concrete compression stresses develop again. The crack closure gradually stiffens the section and transverse instability at that level of loading may be arrested. If buckling does occur, as in specimens CY-1, CY-2 and CY-3, transverse displacements can be restricted only if and when the boundary element can develop an adequate redistributed stress state. This stress state must permit the resistance of internal forces from axial loading on the section and also from transverse moment due to out-of-plane eccentricity. If this state of the stresses is not reached, out-of-plane displacements increase rapidly, even if the internal vertical compression forces are small. This means that after large inelastic displacements, failure by buckling can occur at relatively small axial loads, as happened in these specimens.

In the first four specimens (CY-1–CY-4) significant strength and stiffness degradations were observed. Specimens with smaller transverse reinforcement steel ratios (CY-3 and CY-4) had larger degradations. The observed differences in behavior between CY-1 and CY-2 are mainly due to the magnitude of the deformations imposed, specifically the tensile deformations.

It is clear from the test results that the behavior of the specimens was improved through decreasing the spacing of and/or increasing the size of the BauMesh grids. BauMesh grids made with $\frac{1}{4}''$ diameter provided less confinement under similar transverse reinforcement steel ratios, due to their weakness at the welded connections of the wires.

2.10. Comparison of experimentally observed strengths with those predicted by ACI-318/83

It is interesting to compare the measured resisting loads of the specimens with those expected from the ACI mode. It is usually assumed that the axial strength of the boundary element in compression is given by

$$P = f_y A_s + f'_c(A_g - A_s) \qquad (1)$$

Using design material properties we have

$$P = 60(8 \times 0{\cdot}79) + 4{\cdot}0[(5{\cdot}87 \times 18) - (8 \times 0{\cdot}79)] \quad \text{for CY-1–CY-3}$$

$$P = 776{\cdot}6 \text{ kip}$$

$$P = 60(8 \times 0{\cdot}79) + 4{\cdot}0[(5{\cdot}75 \times 18) - (8 \times 0{\cdot}79)] \quad \text{for CY-4–CY-6}$$

$$P = 767{\cdot}9 \text{ kip}$$

Using measured material properties we have

$$P = 74{\cdot}0(8 \times 0{\cdot}79) + 5{\cdot}2[(5{\cdot}87 \times 18) - (8 \times 0{\cdot}79)] \quad \text{for CY-1–CY-3}$$

$$P = 984{\cdot}2 \text{ kip}$$

$$P = 74{\cdot}0(8 \times 0{\cdot}79) + 5{\cdot}2[(5{\cdot}75 \times 18) - (8 \times 0{\cdot}79)] \quad \text{for CY-4–CY-6}$$

$$P = 973{\cdot}0 \text{ kip}$$

The observed compressive strengths were between 998 and 1 120 kip. These maximum compressive loads correspond to overstrengths between 28·6% and 45·8% if the design material properties are considered, or overstrengths between 1·5% and 15% if the measured material properties are considered.

The tensile capacity of the boundary elements is

$$P = f_y A_s \tag{2}$$

Using design material properties we have

$$P = 60(8 \times 0{\cdot}79)$$

$$P = 379{\cdot}2 \text{ kip}$$

Using measured material properties we have

$$P = 74{\cdot}0(8 \times 0{\cdot}79)$$

$$P = 467{\cdot}7 \text{ kip}$$

The observed tensile strengths were between 520 and 530 kip. These maximum tension loads correspond to overstrengths between 37·1% and 39·8% if the design material properties are considered, or overstrengths between 11·1% and 13·3% if the measured material properties are considered.

The discrepancies between code-estimated and measured resistances, in this case, are primarily due to the differences in the measured mechanical properties of the materials and those assumed in design. This overstrength in axial capacity of the boundary elements may result in an overstrength of the shear wall if the behavior is controlled by flexure. But it is important to mention that if larger or at least equal amount of overstrength are not present in the shear capacity of the wall, a non-ductile shear failure may occur in the wall.

2.11. Conclusions

(i) The deformability of specimens which incorporated BauMesh as transverse reinforcement was highly dependent upon the strength of the resistance grids.

(ii) The mechanical behavior of the specimens incorporating BauMesh grids improved as the size of the wire increased and the spacing decreased. The size and spacing combination of the BauMesh grids that exhibited the most stable mechanical behavior was the combination of $\frac{3}{8}''$ diameter wire modules at 3″ spacing, which corresponded to a transverse reinforcement ratio $\rho_s = 0{\cdot}020$. The confinement effectiveness of BauMesh grids was observed to increase as the transverse reinforcement ratio (the ratio of the volume of transverse steel reinforcement to the volume of confined concrete) increased. An effective means by which to increase the transverse reinforcement ratio is to decrease the spacing of the BauMesh grids.

(iii) To improve the load-strain behavior of reinforced concrete confined either by traditional hoops or by BauMesh grids is of great importance to avoid the premature local buckling of the longitudinal reinforcement. To avoid the premature local buckling of the longitudinal reinforcement, it is required to provide closely spaced transverse reinforcement of a certain minimum size.

(iv) Confinement of the boundary elements of a shear wall significantly improves performance by delaying bar buckling and allowing high compression strains to develop.

(v) Out-of-plane instability can control the behavior of rectangular walls. Premature failures associated with low energy dissipation capacities in the shear walls may occur if cycles of large deformation are imposed on the boundary elements of rectangular cross-section shear walls.

(vi) The strain history of longitudinal reinforcement is a critical parameter related to out-of-plane instability. The most important aspect of strain history during cyclic loading is the magnitude of the tensile strain which occurs prior to compression of the potentially unstable zone. It seems that cyclic loading of small amplitude is not sufficient in itself to induce out-of-plane instability, although further research is needed to be more conclusive in this regard.

(vii) If large inelastic deformations are expected in a structure incorporating shear walls of uniform width, slender walls (i.e. walls that are thin) should be avoided to prevent buckling of the boundary elements of the walls.

2.12. *Recommendations*

(i) The minimum design requirements for BauMesh transverse reinforcement should be in accordance with the 1991 UBC requirements for transverse reinforcement of shear wall boundary elements.

(ii) Other considerations in the design of transverse reinforcement, such as the buckling of longitudinal reinforcement, shear and the improvement of the bonding of longitudinal bars, should be taken into consideration in the design of BauMesh transverse reinforcement for the boundary elements of shear walls.

(iii) The results obtained in the test program cannot be extrapolated to applications where strains in excess of $\varepsilon_c = 0{\cdot}02$ are expected, or where there is an appreciable strain gradient across the section, such as in columns and girders of reinforced concrete moment frames or coupling girders of reinforced concrete frame-wall structures. Neither state of stress and strain nor the boundary conditions present in these cases (e.g. the presence of strain gradient, the presence of shear, etc.) were reproduced in the tests conducted. Further research is needed for such cases so as to ascertain the mechanical behavior of reinforced concrete elements in such conditions.

(iv) In the test program, failure of BauMesh grids occurred at resistance welds or in the heat affected zones near the resistance welds. In no instance was the necking of the wire similar to that observed in tensile tests of the wire observed in the BauMesh grids in the specimens tested in the test program. This was especially observed in specimens CY-4, CY-5 and CY-6, which used BauMesh grids made with $\frac{1}{4}''$ wire. Therefore, since the integrity of the modules and hence the effectiveness of BauMesh grids as transverse reinforcement is dependent upon the development of the nominal yield strength of the wires, it is of paramount importance that proper quality control of the manufacturing process be maintained, such that the strength of the resistance weld is guaranteed to be greater than the tensile strength of the wire. Regular testing of the grids, so as to ensure that the nominal strength of the wire can be obtained prior to the fracture of the weld, should be conducted by the manufacturing facility and be included in quality assurance documentation accompanying the manufacturing records.

(v) If large deformations are expected to be imposed on the shear walls, it is suggested that wherever possible boundary elements thicker than the web should be supplied at the ends of structural walls because of their stabilizing influence (e.g. barbell sections should be provided instead of rectangular sections). These elements are often required anyway from consideration other than stability, e.g. to form a column element in which transverse beams are framed. Although more research is needed in this area, a boundary element width-to-floor height ratio of 1:8 is recommended for the critical wall story (i.e. where large inelastic rotations will develop).

(vi) If the use of enlarged boundary elements is not possible, then the occurrence of large inelastic deformations in these members should be avoided by decreasing the global deformation demands on the structure. A possible way to attain this is to design for larger lateral forces, e.g. use a smaller response modification factor.

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