ASIAN JOURNAL OF CIVIL ENGINEERING (BUILDING AND HOUSING) VOL. 8, NO. 2 (2007)
PAGES 215-225

STUDIES ON RC AND FERROCEMENT JACKETED COLUMNS SUBJECTED TO SIMULATED SEISMIC LOADING

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1. INTRODUCTION

It is known that large inelastic deformation limits of individual members allow entire structures to endorse severe ground motion while dissipating significant levels of seismic energy. Plastic hinge formations associated with lateral displacement excursions is favored in beams and girders rather than in columns to ensure that the overall structural integrity is not compromised. Plastic hinges can occur in columns, however, particularly at the base of multistory frames and bridges where incurred, damage acts to dampen seismic forces considerably. Ductile behaviour is hence essential at these crucial sites to prevent complete structural collapse under sustained loading. The structural response during earthquakes have indicated that the majority of the column failures was caused by high shear stresses, insufficient transverse reinforcement rendering those members ineffective at dissipating seismic energy and inadequate ductility rapidly leading to failure. Typical procedures to compensate for the deficiencies involve external retrofitting of these columns. Recent research works[1,2)] have indicated that ferrocement jacketing may be used as an alternative technique to strengthen RC columns with inadequate shear strength. The present objective is to complement the earlier work of the authors[3,4,5)], on the use of ferrocement jackets for seismic retrofit of non-ductile reinforced concrete columns with inadequate shear strength. The response of R.C and ferrocement retrofitted columns to seismic loading was examined under three different axial load ratios.

Keywords: ductility, shear strength, ferrocement, energy absorption capacity

2. RESEARCH SIGNIFICANCE

The research work forms a part of experimental investigations aimed at developing an efficient and economical method of retrofitting existing reinforced concrete structures for enhanced shear resistance. The shear strength of reinforced concrete members under inelastic loading is affected by a number of parameters including the axial load ratio. This research is aimed at examining the effect of axial load on the hysteretic response and energy absorption capacity of RC and ferrocement confined columns. The results can be further

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used in developing the design guidelines for retrofitting with ferrocement.

3. EXPERIMENTAL PROGRAM

The experimental program consisted of three scale model bridge pier specimens designed as shear deficient specimens, tested under different axial loads, before and after retrofitting with ferrocement jackets. The specimens were reinforced with 6 bars of 16mm diameter distributed evenly around the perimeter of the pier cross section. 6 mm diameter ties at 300mm spacing were used as the transverse reinforcement. Ordinary Portland cement with a specific gravity of 3.14 and natural sand passing through JIS sieve 2.5(2.36mm) were used for the core concrete. The test specimens were cast in the vertical position. To minimize the differences in the concrete compression strength among the specimens, all the columns were cast with the same batch of concrete and on the same day. A number of 100 x 200 mm cylinders were cast for each batch to estimate the compressive strength. After about four days of casting, the specimens were covered with damp burlap to prevent moisture loss. The specimens were stripped off 7 days after casting and then air cured before testing.

3.1 Ferrocement Jackets

The three RC columns were strengthened with six layers of ferrocement jackets (V_f =3.46%) after their failure. The volume fraction (V_f) is the ratio of the volume of the mesh reinforcement to the volume of the composite (ACI Committee 549[6]). Woven wire mesh of 2.76mm square opening and 0.44mm diameter were used as reinforcement for ferrocement jackets through out the test program. The average yield strength based on 0.2% permanent strain was found to be 597 MPa. The properties of the ferrocement plates under tensile load can be determined as per the procedure suggested by ACI Committee 549[6]. The specimen used was 500mm long, 70mm wide, 12mm thick, and symmetrically reinforced. The details of the strength of the mesh wire and ferrocement plates are given in Table 1.

Table 1. Material property of mesh and ferrocement plates

Diameter of mesh wire (mm)	**Grid Spacing of mesh wire (mm) Longitudinal**	**Transverse**	**Yield Strength of mesh wire (MPa)**	**Ultimate Strength of mesh wire (MPa)**
0.44	2.76	2.76	597	706

Measured Tensile Strength of Ferrocement Plates

S. No	No. of Layers	Volume Fraction (% V_f)	Tensile Strength (MPa)	Maximum Stress (MPa)
1	4	2.94	8.68	11.42
2	6	3.46	11.84	15.14

The required width of the mesh, approximately 985mm and the length as per the number of layers, was cut and properly wrapped around the skeletal steel continuously through out the column. At several places, the different layers of the wire mesh were tied together with the same diameter of steel wire (Figure 1). An overlap of 100mm was provided in the lateral direction for the wire mesh. A clear cover of 2.5mm on the inner most and outer surfaces of the wire mesh was provided using spacer rods. The infill mortar was made with a rich mortar matrix 1:1.20 with 0.40 water/cement ratio. Natural sand passing through JIS sieve No. 1.2(1.18mm) was used.

Figure 1. Ferrocement jacketing

Figure 2. Spacer rods

As rich mortar matrix was used, there were workability problems. This was overcome by addition of a water-reducing admixture in the mortar matrix in optimum dosages as a percentage by weight of cement. The specimens were cast on the same day using identical mortar mix proportion, so that there was no change in the strength. A number of 50mm diameter mortar cubes were cast with the same mortar as used for ferrocement jackets. The 7-day strength of core concrete was 24Mpa and mortar strength was 30Mpa. To ensure that the mortar penetrated properly through the inner layers of ferrocement jacket, temporary spacer rods of 2.5mm diameter were provided between the mesh layers (Figure 2). Shrinkage compensating high performance mortar was injected between the concrete surface and the ferrocement jacket and in between the layers. A small gap of 15mm was provided at the bottom of piers between the ferrocement jacket and the bottom of the footing. This prevents excessive increase in the flexural strength. To increase the flexural strength of columns in a controlled manner, anchor bolts were provided at the bottom of the ferrocement jacket in the foundation portion. The appropriate number and the size of the bolts were selected in such a manner that the degree of increase of the flexural strength was controlled. The gap provided between the jacket and footing was required to trigger the failure. Care was also taken to prevent the bulging of the longitudinal bars and maintain the confining effect of the jacket.

3.2 Test Procedure

Figure 3 shows the test setup. The test specimen was a cantilever with the fixed end framing into a footing. The specimen was intended to approximate one third of a column in a real bridge frame. All the specimens were subjected to a cyclic lateral load at the tip of the cantilever. The loading test was performed with a dynamic shaker installed at the head of the pier. Regarding loading hysterisis the displacement when the tensile reinforcement in the axial direction of a pier yielded (yielding displacement) was considered as a unit[7,8,9]. Specific times of positive and negative loading are repeated alternatively with displacement amplitude of an integer multiple of the unit and displacement amplitude was gradually increased. In the present case, in the initial stages,

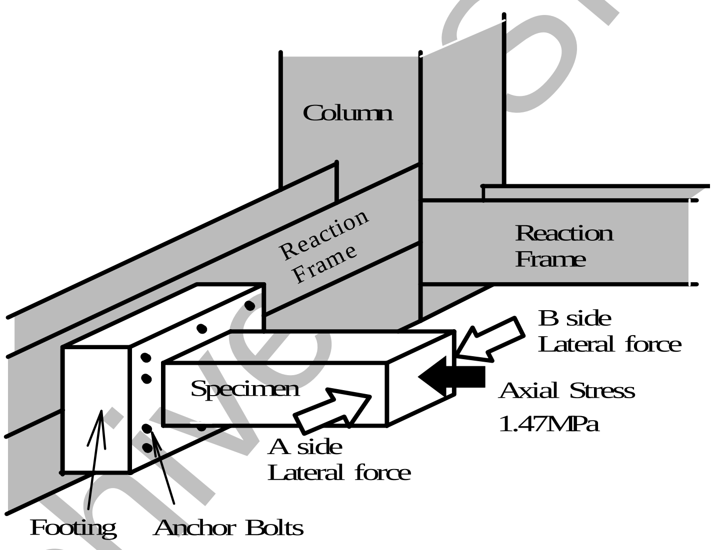

Figure 3. Schematic diagram of test set up

The lateral load was increased under load control in increments of 10kN until the yielding displacement reached and after this displacement control in increments of yielding displacement were applied. The three RC columns were identical in respect of cross section and reinforcement. The main variable parameter in the study was the axial load ratio[10,11]. Cyclic lateral loading was applied on the scale model test piers while being simultaneously subjected to axial loads of 100, 150 and 200kN respectively. The foundation portion of the specimen was securely tied to the reaction floor beam of the frame by tightening with bolts, while the top was free to move without inducing any rotation. The lateral load was applied by a 200mm stroke, electrically operated accelerometer with a capacity of 250kN. The specimens were instrumented with strain gauges mounted in the longitudinal and lateral reinforcement at appropriate positions. Displacement transducers were installed horizontally and vertically along the column enabling measurement of lateral and longitudinal displacements. Data Acquisition system with a 16-channel card scope and 16-channel signal conditioner devices was used for monitoring and analyzing the collected data.

4. BEHAVIOUR UNDER LATERAL LOAD

4.1 Un-strengthened Specimens

The basic un-strengthened columns S1, S2 and S3 exhibited shear failure. Figure 4 shows the hysteretic response of a typical RC column subjected to a constant axial load of 100kN. Flexural cracks were developed close to the bottom end of the columns perpendicular to the column axis in the initial loading cycles. With increase in the displacement, the additional cracks formed and the failure finally resulted by widening of a major inclined crack formed at the lower part of the pier.

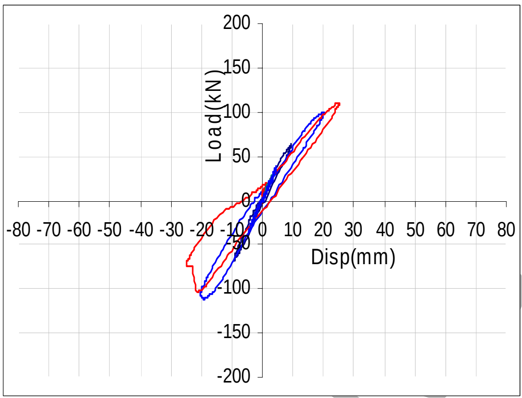

Figure 4. Load-displacement diagram (S1)

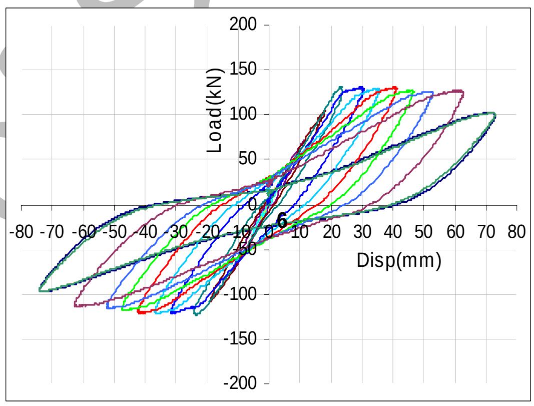

Figure 5. Load-displacement diagram (F1)

4.3 Envelopes of Cyclic Response

Figure 6 and Figure 7 show the cyclic response of the un-strengthened and strengthened columns respectively. The basic un-retrofitted columns S1, S2 and S3 exhibited poor behaviour. The columns F1, F2 and F3 exceeded their flexural capacity, reached higher strengths and exhibited large ductility. The envelopes clearly demonstrate that thin rectangular ferrocement jackets can significantly improve the strength, ductility and energy dissipation of rectangular columns with inadequate shear strength. Although the ferrocement jackets were terminated 15mm from the bottom of the column to prevent an increase in flexural capacity, the retrofitted columns showed significant increase in the flexural capacity and consequently an increase in the shear demand. The increase in the flexural strength may be due to strain hardening of the longitudinal steel reinforcing bars and increase in core concrete compressive strength due to confinement provided by ferrocement jacket.

A comparison of the three RC specimens indicated that for the same design strength and reinforcement the rate of stiffness degradation with respect to the yield displacement was higher for higher axial loads. In case of ferrocement specimens the specimen with 200kN constant axial load (F3) had higher initial stiffness, but the degradation of flexural strength was more sudden as compared to the other two specimens (F1 and F2), which seemed to be more stable. The effect of axial compression on column response was the acceleration of strength and stiffness degradation under repeated inelastic load cycles.

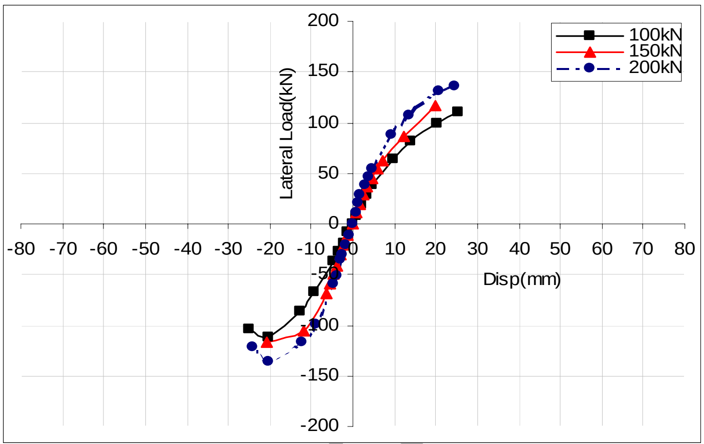

Figure 6. Load-displacement envelopes (RCC)

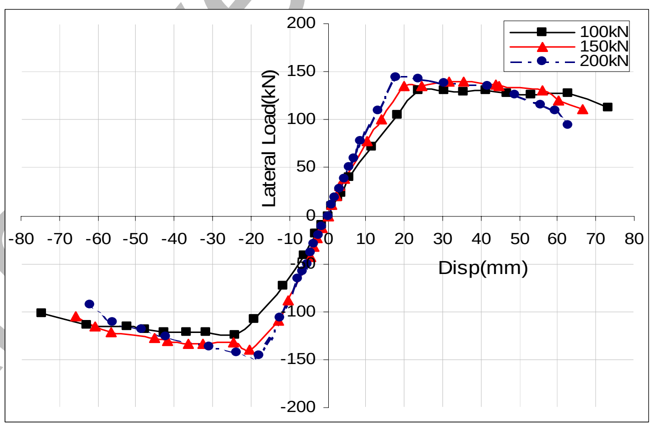

Figure 7. Load-displacement envelopes (FCC)

5. COMPARISON OF DEFLECTION COMPONENTS

The columns were well instrumented with laser devices and linear gauges to measure the components of horizontal displacement at different heights of the specimen. Figs-8 and 9 show the lateral deflection of the column before and after retrofitting with ferrocement at different heights for different lateral load increments. Figure 10 shows a typical comparison of the maximum displacements at different heights for all the un-strengthened and strengthened columns subjected to identical axial compression. It may be noted that there is a linear increase in the deflections along the height of the column.

6. STRAINS IN LONGITUDINAL REINFORCEMENT

Figures 11 and 12 show the typical load-longitudinal strain relationship of RCC and ferrocement specimens respectively for the axial load

100kN.

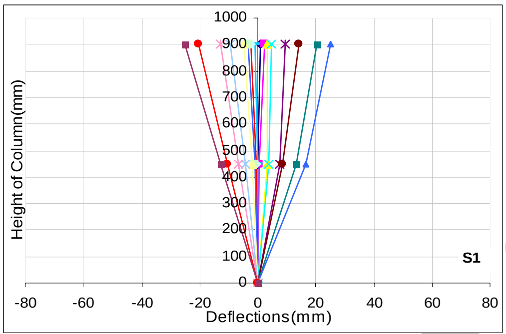

Figure 8. Deflection components (S1)

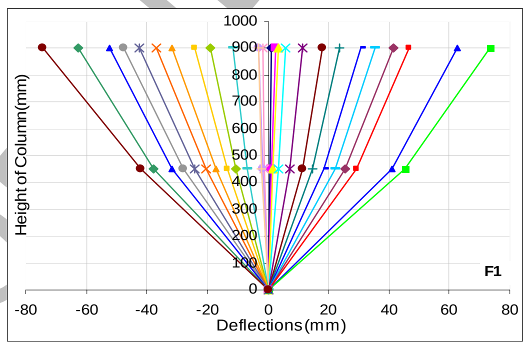

Figure 9. Deflection components (F1)

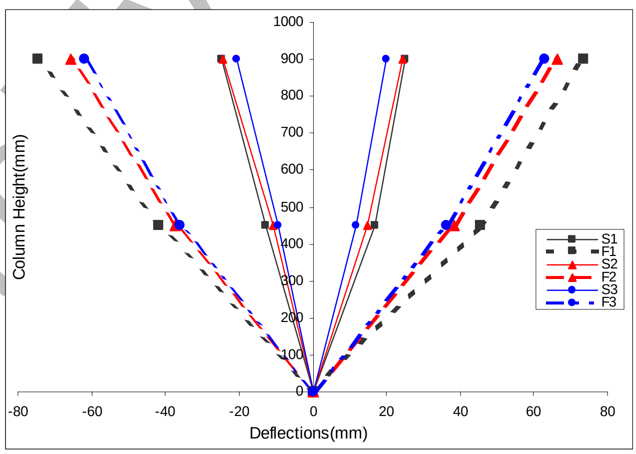

Figure 10. Comparison of deflection components

A comparison of the strain response of un-strengthened and strengthened specimens at the middle height of the specimen indicated that the longitudinal bars have deformed. In case of strengthened specimens the ferrocement jacket has also yielded which was evident for higher strain values as compared to RCC specimens. The results indicated that the ferrocement specimens started taking the loads beyond the yield point of the reinforcing rods in the post-elastic region enabling a stable hysterisis. This indicates the ductile behaviour of ferrocement specimens. This was true for the three axial loads 100, 150 and 200kN.

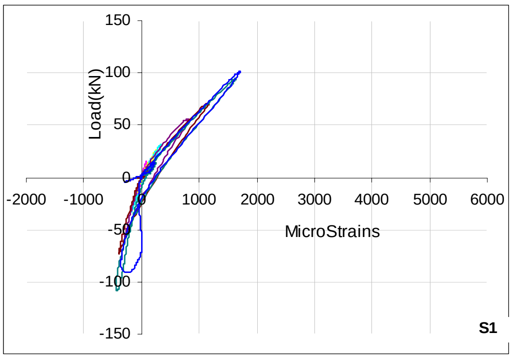

Figure 11. Load vs longitudinal strain (S1)

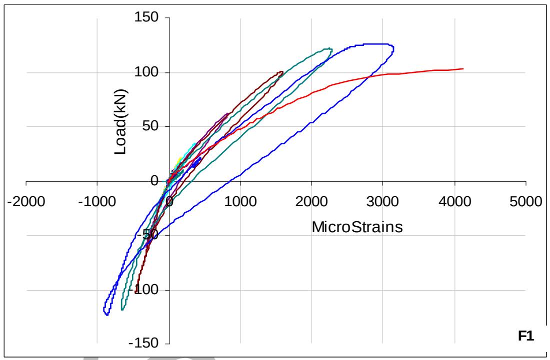

Figure 12. Load vs longitudinal strain (F1)

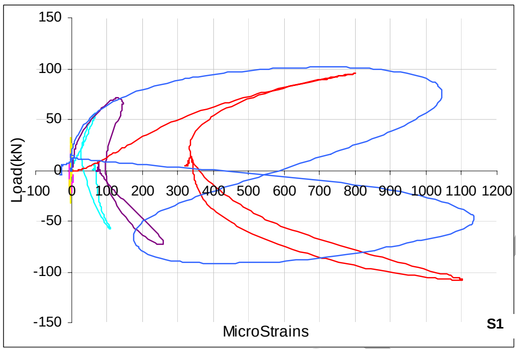

Figure 13. Load vs Lateral Strain (S1)

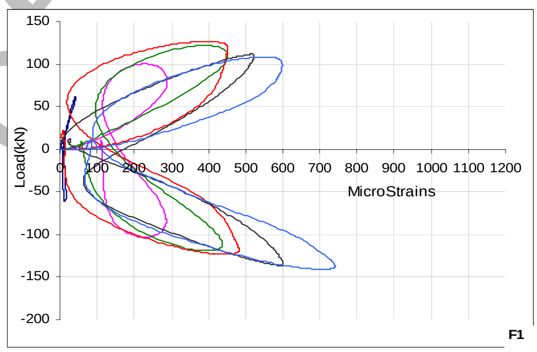

Figure 14. Load vs Lateral Strain (F1)

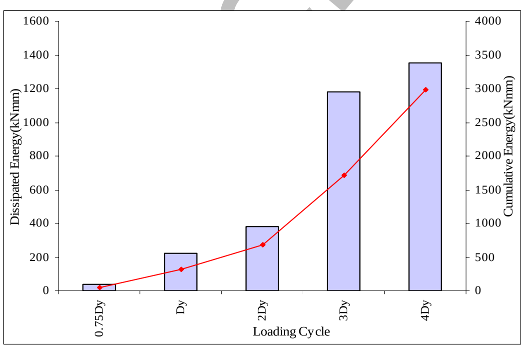

Figure 15. Energy dissipation (S1)

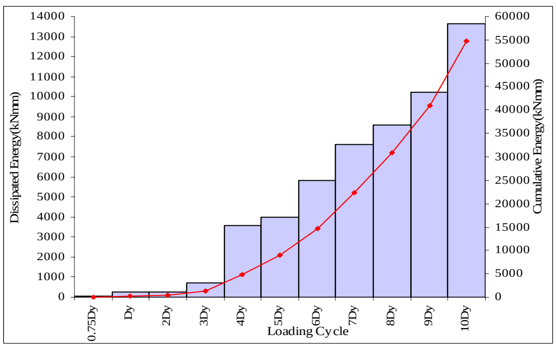

Figure 16. Energy dissipation (F1)

7. STRAINS IN TRANSVERSE REINFORCEMENT

The strains in the transverse reinforcement of the basic un-strengthened columns reached high levels while remaining below yield in the un-strengthened columns, even though the strengthened columns were loaded to higher lateral shear forces. The behaviour was same for all the strengthened columns loaded at different axial loads. Figures 13 and 14 show typical plots of lateral strains for the basic un-retrofitted and ferrocement retrofitted specimens respectively. The strains corresponding to un-strengthened columns were high and experienced shear failure. On the other hand in case of ferrocement specimens the lateral strains were much lower at identical displacements. The response can be attributed to the fact that the presence of ferrocement jackets did not allow the major diagonal shear cracks to open, even though the columns were loaded to large lateral displacements.

8. ENERGY DISSIPATION

8.1 Calculation of Dissipated Energy

The energy dissipated by a test specimen is defined as the area enclosed by the hysterisis loop of the lateral load displacement relationship for each individual cycle with units in kN-mm. The calculation was carried out by a spreadsheet, numerically adding up the area in a loop for each completed loading cycle, up to the cycle before the specimen reached the ultimate or failure state.

8.2 Individual Cycle Energy

Figure 15 shows the dissipated energy by each individual loading cycle and the accumulation of the dissipated energy of the specimen S1. The observation was similar in case of all the three specimens. Figure 16 represent the dissipated energy and the cumulative energy for ferrocement specimen F1. The energy dissipation increased with the displacement and this was true for all axial load cases in case of ferrocement specimens also. It was noted that with increase in the axial load the compression area was more as compared to tension area.

8.3 Total Dissipated Energy

Figure 17 shows the total dissipated energy of all the RCC and ferrocement until failure. It is very much evident that ferrocement specimens exhibited a tremendous energy dissipation capacity, very important for earthquake resistance. It was also observed in case of ferrocement specimens that the cumulative energy dissipated energy has decreased, though marginally, with increase in the axial load ratio.

9. CONCLUDING REMARKS

The external confinement using ferrocement resulted in enhanced stiffness, ductility, strength and energy dissipation capacity. The mode of failure could be changed from brittle shear failure to ductile flexural failure. The axial loads influence the hysteretic response of columns and the energy absorption capacity. The effect of axial compression on column response was the acceleration of strength and stiffness degradation under repeated inelastic load cycles.

Acknowledgements: The authors wish to thank the Department of Civil Engineering, Kitami Institute of Technology, Japan and Monbusho-Mext for providing the necessary support in conducting the experimentation.

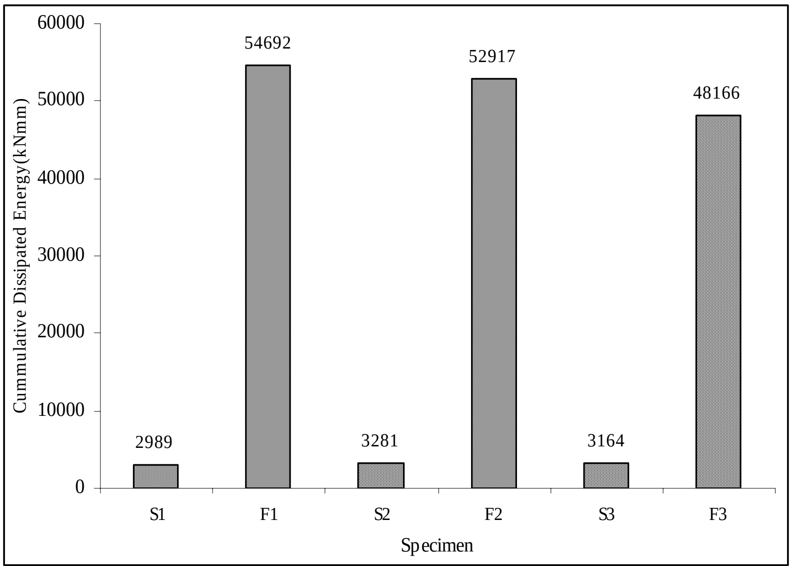

Figure 17. Total dissipated energy

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