Zeitschrift: IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen

Band: 14 (1973)

Artikel: Cyclic load tests on concrete frames

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DOI: https://doi.org/10.5169/seals-14479

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Cyclic Load Tests on Concrete Frames

Essais de charge cyclique sur cadres en béton armé

Zyklische Belastungsversuche an Stahlbetonrahmen

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1. INTRODUCTION

Tests on concrete frames subjected to cyclic (reversed) loading are at present being conducted at the University of Canterbury. Full scale beam-column assemblies with a range of proportions of prestressing steel and ordinary reinforcing steel are being tested to examine the behaviour of plastic hinge regions in members and joint cores. The tests are aimed at determining the deformation capacity and degree of damage of concrete frames when responding to severe earthquake motions and will provide information for design. The results from three beam-column assemblies, and a repaired assembly, subjected to static cyclic loading will be briefly described.

2. TESTS ON FULL SCALE BEAM-COLUMN UNITS

2.1 Test Specimens and Loading

The dimensions of the three test units are shown in Fig. 1. The test units represented approximately that part of a frame between points of contraflexure at a typical interior beam-column joint. The loading was as shown in Fig. 2 with an axial column load P = 224 kips (996 kN) and transverse loads on the ends of members representing the shear forces induced by earthquake loading.

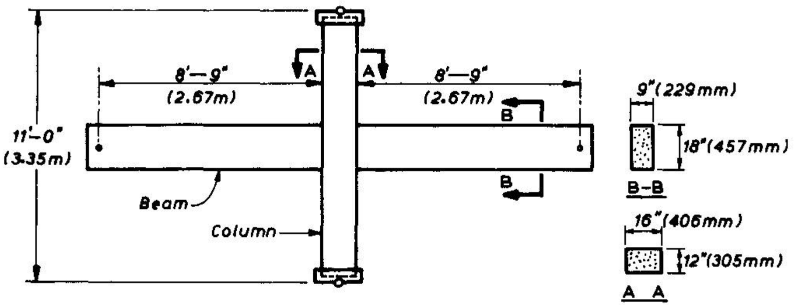

Fig. 1 Dimensions of Units 1, 2 and 3
(1" = 25.4 mm)

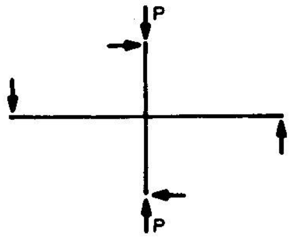

Fig. 2 Loading System

By reversing the directions of the transverse loads the effect of earthquake shaking was simulated. The transverse loading was such as to enforce deformations into the inelastic range several times. Plastic hinging was expected to occur in the beams because the flexural strength of the columns was greater than that of the beams. Fig. 3 shows a test unit at the start of testing.

Details of the cross sections of the columns and the beams are shown in Figs. 4 and 5. The beams of the test units contained various quantities of prestressed high tensile steel and non-prestressed mild steel deformed bar. Unit 1 had a fully prestressed beam, Unit 2 had a partially prestressed beam and Unit 3 had an ordinary reinforced beam. All beams had approximately the same theoretical ultimate flexural strength. The columns of all three test units were identical, containing non-prestressed high strength steel deformed bars in the longitudinal direction. The mild steel shear reinforcement in the members was designed according to the ACI building code[1]. In the beams the stirrups were sufficient to carry all the shear force. In the columns some shear was assumed to be carried by the concrete. In the joint region $\frac{5}{8}$ in (16 mm) diameter hoops at 2 in (51 mm) centres were provided to carry the

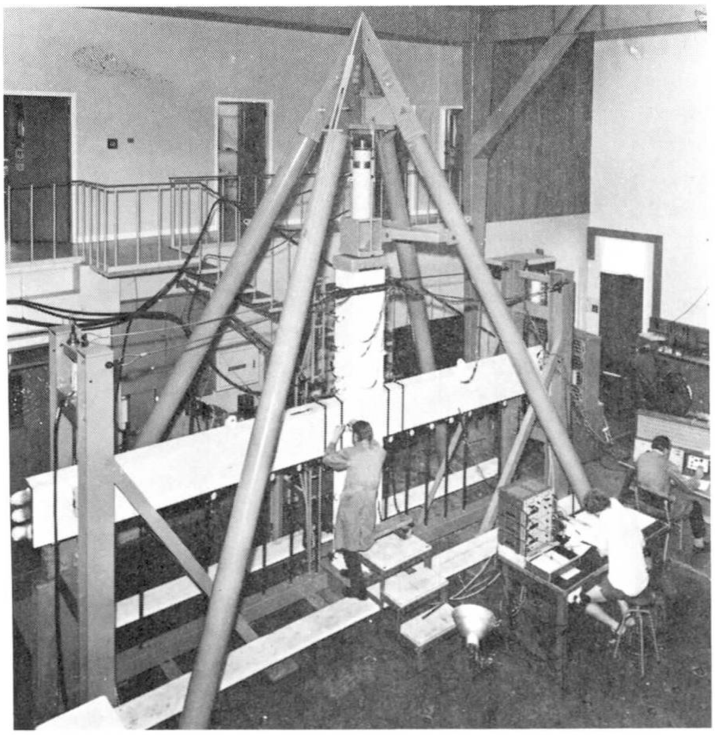

Fig. 3 Unit 1 in Test Frame at Start of Testing.

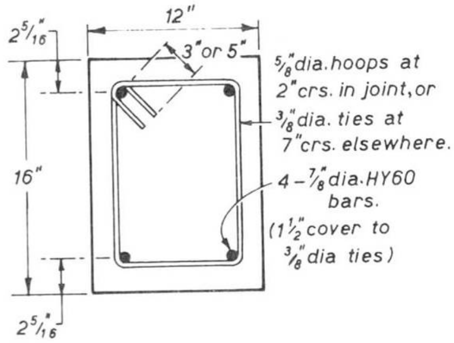

Fig. 4 Column Section of Units 1, 2 and 3 (1" = 25.4 mm).

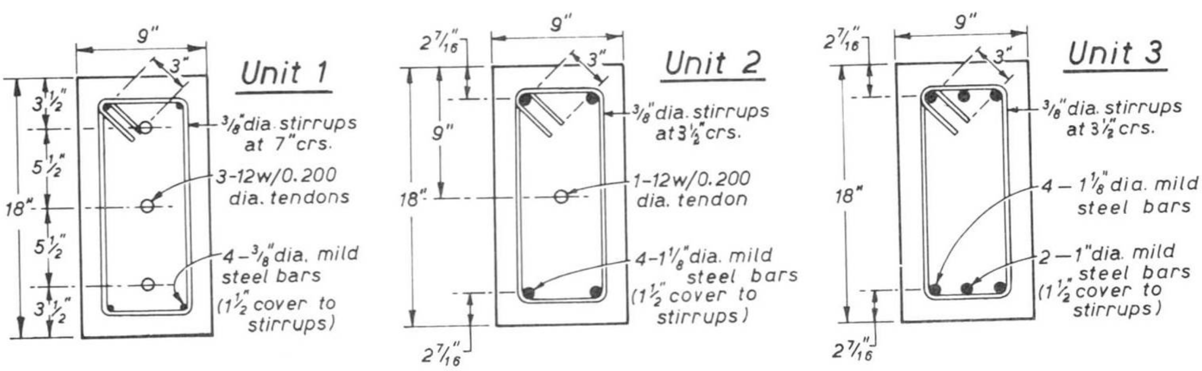

Fig. 5 Beam Sections (1" = 25.4 mm)

horizontal shear forces acting on the joint core due to the column shear and the tensile and compressive forces in the beams acting on each side of the joint core.

The concrete compressive cylinder strength for the members at the time of testing ranged between 4.63 ksi (31.9 N/mm^2) and 5.51 ksi (38.0 N/mm^2). The prestressing steel had an ultimate tensile strength of 236 ksi (1628 N/mm^2). The prestressing tendons were post-tensioned to 70% of the ultimate tensile strength and grouted. The yield stresses of the reinforcing steel were 40.4 - 46.0 ksi (279 - 317 N/mm^2) for the longitudinal beam steel, 58.8 - 60.2 ksi (406 - 415 N/mm^2) for the longitudinal column steel and 41.9 - 43.5 ksi (289 - 300 N/mm^2) for the transverse steel.

2.2 Test Results of Original Units

Figs. 6, 7 and 8 show for the units the vertical deflection at the ends of the beams plotted against the bending moment in the beams at the column face, and views of the test units showing the degree of damage sustained at two load stages.

In the three test units the maximum moments reached in the beams in the initial inelastic loading runs in each direction were within 9% of the theoretical flexural strengths of the beams calculated for an extreme fibre concrete compressive strain of 0.003 from first principles using the stress-strain curves for the steel and concrete, strain compatibility across the section and equilibrium of internal forces. With subsequent load cycles in the case of the prestressed concrete beam the inelastic deformation and damage concentrated in the beam and the joint core remained intact, whereas in the case of the partially prestressed and reinforced concrete beams the inelastic deformation and damage concentrated in the joint core and the adjacent members remained intact.

For Unit 1 (prestressed beam) the decrease in the strength and stiffness of the beam with further cyclic loading was due to the reduction of the beam cross-section caused by crushing of the concrete. This pointed to the need for more confinement of the concrete by transverse steel at closer spacing. The energy dissipation of the prestressed member was small until concrete crushing had commenced. In this unit the inelastic deformations were mainly due to the plastic hinge rotation in the beams, as is evident from the views of the tested unit shown in Fig. 6.

For Unit 2 (partially prestressed beam) and Unit 3 (ordinary reinforced beam) the failure concentrated in the joint core. The attainment of the flexural strength of the beams of both of these units was accompanied by severe diagonal tension cracking in the region of the joint core, particularly in the case of Unit 3, as is evident from the views of the tested units shown in Figs. 7 and 8. With further loading cycles the maximum beam moment reduced owing to a reduction in the shear strength of the joint cores of these units and the deflection at the end of the beams became mainly due to shear deformations of the joint core. In these two units the longitudinal reinforcement in the beams also showed some tendency to slip backwards and forwards through the joint core due to breakdown of bond, causing a reduction in stiffness of the units.

Comparison of the moment-deflection curves of Figs. 6, 7 and 8 indicates that even after large inelastic deflections the prestressed concrete beam had great ability to recover deflections. Nevertheless the energy dissipated by the prestressed concrete unit (as represented by the area within the hysteresis loops) was quite large after crushing of concrete had commenced. The energy dissipated by the partially prestressed and ordinary reinforced units was larger due to the presence of ordinary steel reinforcement.

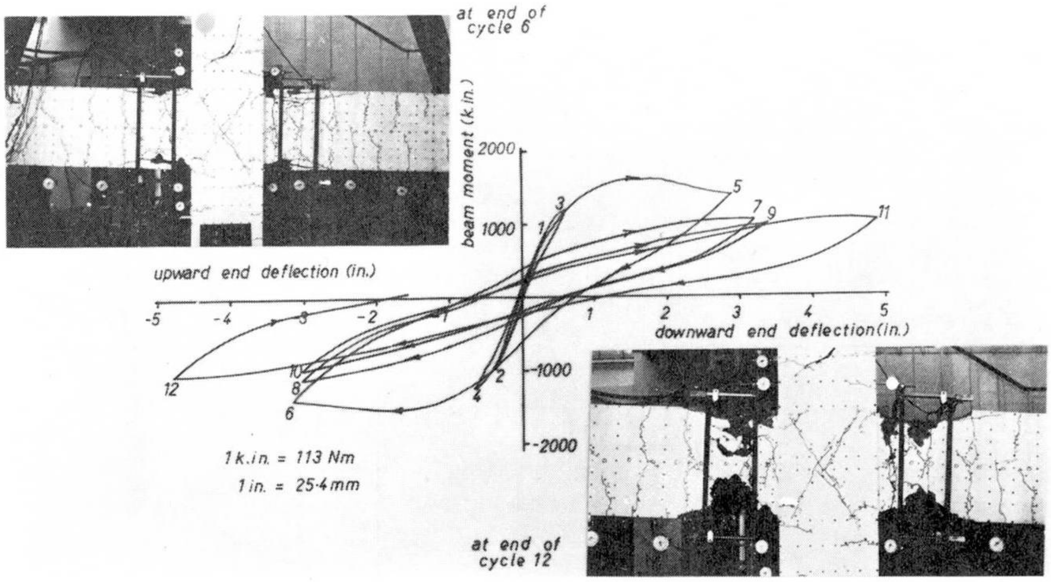

Fig. 6 Unit 1 Beam Moment at Column Face versus Beam End Deflection

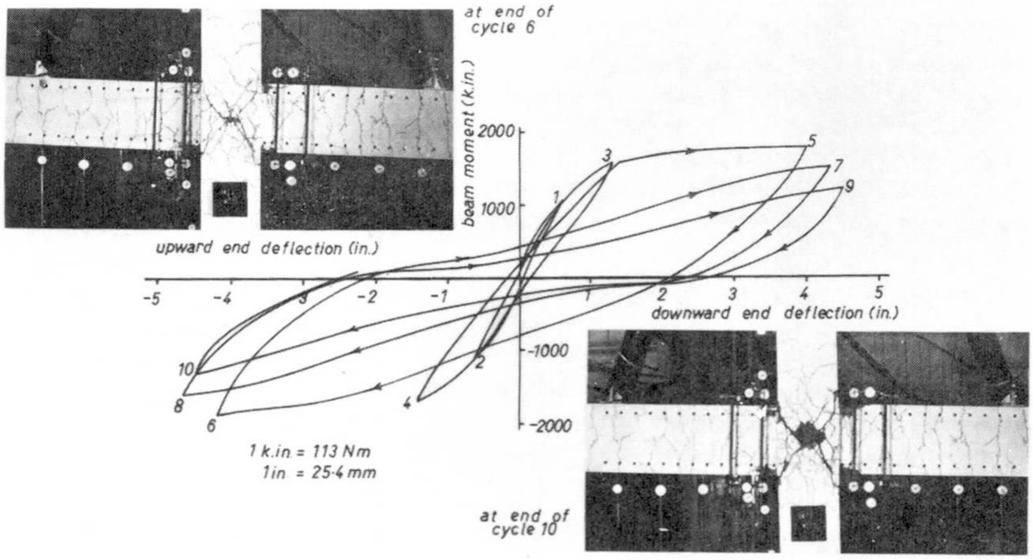

Fig. 7 Unit 2 Beam Moment at Column Face versus Beam End Deflection

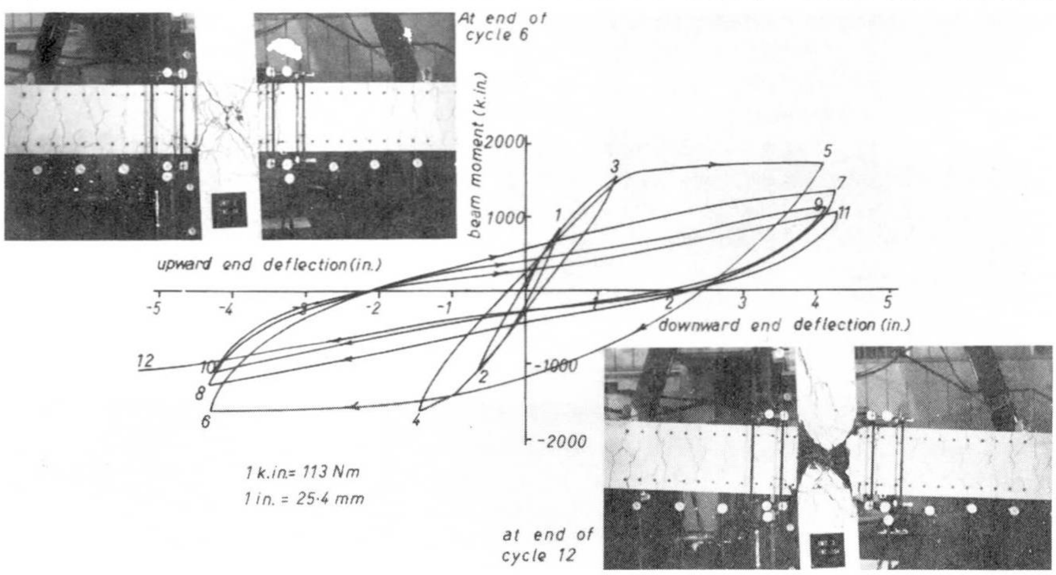

Fig. 8 Unit 3 Beam Moment at Column Face versus Beam End Deflection

2.3 Test Results of Repaired Unit 1

After the testing of Unit 1 it was decided to repair the damaged concrete and to retest the unit to investigate whether satisfactory structural behaviour could be achieved. The damaged beam was repaired by straightening the beam, chipping away the damaged concrete, placing two new stirrups around the remaining core of the beam in each damaged region and casting new concrete to give the original cross-sectional dimensions. Fig. 9 shows the new concrete placed at one beam region and the other region ready for the mould to be placed around for the new concrete. The new concrete had a maximum aggregate size of $\frac{3}{8}$ in (9.5 mm) and at the time of testing the unit had a compressive cylinder strength of 7.40 ksi (51.0 N/mm^2).

The repaired Unit 1 was subjected to similar load cycles as the original unit. Fig. 10 shows the vertical deflection at the end of the beam plotted against the bending moment in the beam at the column face, and the damage sustained at two load stages. Comparison with Fig. 6 shows that the initial stiffness of the repaired unit was less than that of the original unit. This would have been due to the early cracking of the new concrete and the presence of cracks in regions of the original concrete. It is to be noted that the new beam concrete was not prestressed, the tendons merely acting as ordinary reinforcement in that concrete. The crack control in the new concrete was not as good as in the original unit but the cracking was not excessive. For example, at 63% of the measured ultimate moment the maximum crack width in the new concrete was 0.015 in (0.38 mm) whereas in the original prestressed beam that crack width was not reached until 98% of the measured ultimate moment had been applied. The maximum measured bending moments reached in the beams in the repaired unit in the initial inelastic loading runs in each direction were within 14% of the maximum measured bending moments of the original unit. The joint core remained undamaged apart from small diagonal tension cracks.

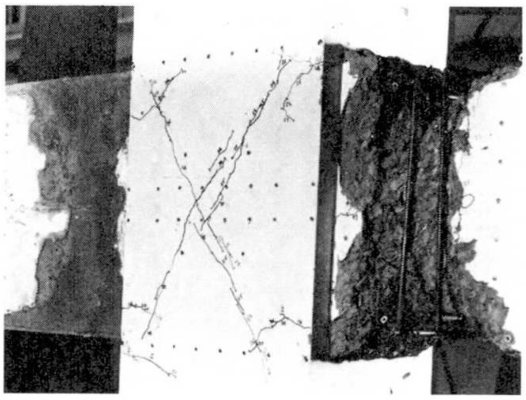

Fig. 9 Repair of Unit 1

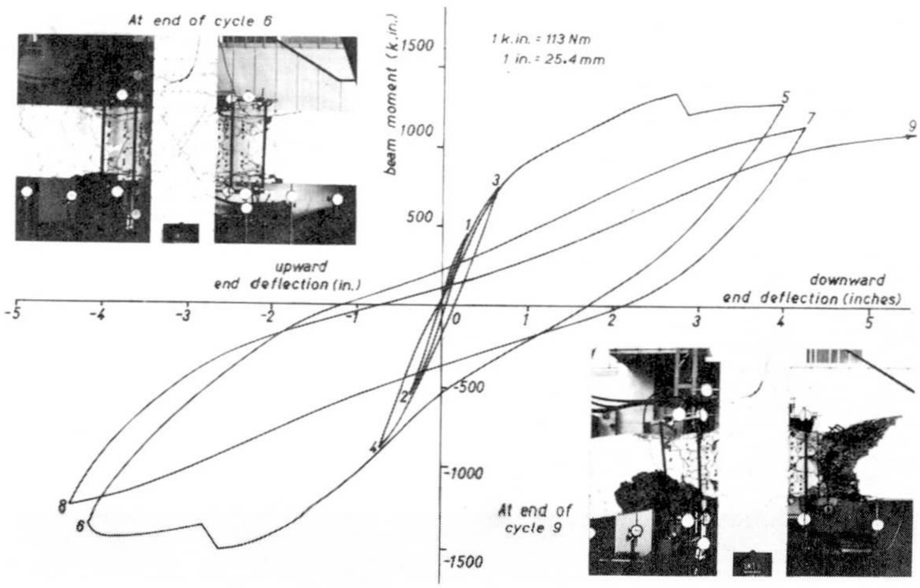

Fig. 10 Repaired Unit 1 Beam Moment at Column Face versus Bend End Deflection

3. CONCLUSIONS

The main findings from the tests are:

(a) Beams: In order to obtain ductile behaviour of prestressed concrete beams under intense inelastic reversed loading, adequate confinement of concrete should be provided by transverse steel in plastic hinge regions to prevent excessive loss of section when crushing of concrete commences. The exact amount of transverse steel necessary requires further investigation but tentatively it appears that a stirrup spacing of 4 in (100 mm) should not be exceeded in these regions, and closer spacing may be necessary. The partially prestressed and ordinary reinforced beams showed good ductile behaviour mainly due to the presence of compression reinforcement. However the compression reinforcement showed a tendency to slip through the joint core, due to breakdown of bond, thus reducing its effectiveness.

(b) Columns: In these tests the columns were stronger than the beams and hence were not critical elements, apart from the joint core regions.

(c) Joint Cores: The joint was not critical in the assembly with the prestressed beam. However in the assemblies with the partially prestressed beam and the ordinary reinforced beam a shear failure occurred in the joint core in spite of shear reinforcement being placed there according to the design procedure of ACI 318-71[1]. In the prestressed concrete beam the compressive concrete force was higher and this evidently helped the joint core behaviour, along with the presence of the central prestressing tendon. Aspects of joint core behaviour are not fully understood and require much further investigation.

(d) Energy Dissipation: All units showed considerable energy dissipation once the maximum moment capacities had been reached. The ordinary reinforced assembly showed greater energy dissipation than the assembly with the partially prestressed beam which in turn showed greater energy dissipation than the assembly with the prestressed concrete beam. However comparisons are difficult because for the assembly with the prestressed beam the inelastic deformation came mainly from the beam plastic hinges whereas for the other assemblies the inelastic deformation eventually came mainly from the shear deformations of the joint cores.

(e) Repair: Repairs made to the prestressed beam by replacing the damaged concrete, and the retesting, showed that it was possible to repair badly damaged members. Repairs to the assemblies with damaged joint cores would have been much more difficult however.

Acknowledgements

The tests reported were carried out by graduate student K.J. Thompson and technicians A.G. Foot and J.M. Adams under the supervision of the author.

References

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-71)", American Concrete Institute, Detroit, Michigan, 1971, pp. 78.

SUMMARY

Tests on concrete frames subjected to static cyclic loading simulating the effect of earthquake shaking are described. The test frames were full-scale interior beam-column assemblies with a range of proportions of prestressing steel and ordinary reinforcing steel. The tests are part of a continuing series aimed at determining the deformation capacity and degree of damage of concrete frames when responding to severe seismic load reversals. The results from three test frames are given, including the behaviour of a frame retested after repairing damage. The importance of traverse steel for confinement of compressed concrete in members, and for shear reinforcement in joint cores, is emphasized.

RESUME

On décrit des essais effectués sur cadres en béton armé soumis à une charge cyclique statique due à l'effet de secousses aux tremblements de terre. Les cadres d'essai étaient composés de poutres et colonnes à l'échelle, en acier précontraint et en acier d'armature habituel. Les essais font partie d'une série pour la détermination du degré de déformation et des dégats aux cadres en béton armé soumis à des secousses alternées. On montre les résultats obtenus sur trois cadres d'essai y compris le comportement d'un cadre aux essais exécutés après l'élimination des dégâts. L'importance de renforts transversaux servant à limiter le béton comprimé dans les membres et de renforts de cisaillement dans les noyaux est soulignée.

ZUSAMMENFASSUNG

Es werden Versuche an Stahlbetonrahmen beschrieben, die einer statischen zyklischen Belastung hinsichtlich der Wirkung von Erdbebenerschütterungen ausgesetzt sind. Die Testrahmen waren massstäbliche innen zusammengebaute Balken und Stützen mit proportionalen Anteilen an vorgespanntem und gewöhnlichem Armierungsstahl. Die Versuche sind Teile einer fortlaufenden Reihe zur Bestimmung der Grösse der Deformation und der Höhe der Schäden an Stahlbetonrahmen, wenn diese stark wechselnder Erdbebenbelastung unterworfen sind. Es werden die Resultate an drei Versuchsrahmen, einschliesslich des Verhaltens eines Rahmens bei nachträglich vorgenommenem Versuch nach Behebung des Schadens mitgeteilt. Die Wichtigkeit von Querstreben zur Begrenzung komprimierten Betons in Baugliedern und von Schubbewegungen in Kernen wird hervorgehoben.

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