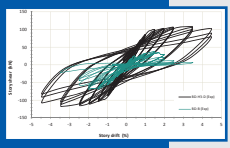
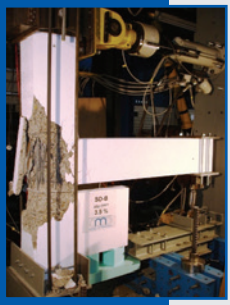
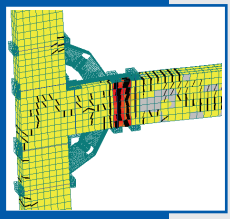
Nº. 9

Leibniz Universität Hannover

Berichte des Instituts für Massivbau







Mahdi Hayatrouhi

Strengthening of reinforced concrete beam-column joints to increase seismic resistance

Fraunhofer IRB Verlag

Berichte

des Instituts für Massivbau der Leibniz Universität Hannover
Herausgeber:
Univ.-Prof. Dr.-Ing. Steffen Marx
Leibniz Universität Hannover – Institut für Massivbau
Heft 9, Februar 2014

Fraunhofer IRB Verlag

Herausgeber
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Leibniz Universität Hannover
Institut für Massivbau
Appelstraße 9 A
30167 Hannover

ISBN (Print): 978-3-8167-9177-5
ISBN (E-Book): 978-3-8167-9178-2

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URL http://www.baufachinformation.de

Abstract

Current research attempted to explore the behaviour of critical regions of reinforced concrete frame structures under seismic loading to investigate the deficiencies and evaluate the performance of gravity load designed (GLD) reinforced concrete (RC) beam-column joints. The categorized literature review of retrofitting and strengthening methods of RC beam-column joints clarified that non-disruptiveness; practical implementation, ductility and perseverance of lateral resistance as well as economical issues still remain the most challenging aspects of seismically retrofitting the vulnerable existing RC beam-column joints.

The seismic design principals of RC frame structures were observed in seismic retrofitting of the vulnerable frames as a strategy of retrofitting based on the capacity design concept. Accordingly, the beam sidesway mechanism was redefined for seismic retrofitting by relocating the beam plastic hinges far enough away from the joints. Afterwards, with introducing innovative energy dissipation devices such as Multi Functional Corbels (HMFC) and Harmonica Damper Plates (HHDP), the innovative Retrofitting Techniques 1 and 2 (RT1 and RT2) were proposed. The introduced devices of HMFC and HHDP as a passive energy dissipation system absorb energy through inelastic deformations. For efficiently and extensively evaluating and arranging the anticipated hierarchy of strength in beam-column joints before and after retrofitting, the Strength and Failure Sequence Diagram (SFSD) was proposed in a new coordinate. To implement the proposed RT1 and RT2 and achieve the desired hierarchy of strength, the design procedures were presented. Subsequently, to clarify the behaviour and founding the proposed innovative devices and techniques a comprehensive numerical analysis was carried out by nonlinear finite element analysis software ATENA.

The proposed RT1 and RT2 were experimentally evaluated through a series of five 3/4-scale beam-column joint specimens including two units for reference and the three others for retrofitting. A particular loading setup was designed and fabricated in structural laboratory so that the applying of horizontal cyclic and vertical static loads became simultaneously possible. An extremely severe loading history including three cycles (push and pull) at every particular drift level as a displacement-controlled series of progressively increasing displacement amplitudes in accordance with [ACI 374.1-05] was imposed to every specimen. The excellent performance of retrofitted specimens through the experimental study confirmed that the proposed RT1 and RT2 are able to retain structural integrity with the minimum strength and

stiffness degradation. As intended, the energy dissipation capacity was dramatically increased and beam sidesway mechanism was actually formed.

Finally, non-linear finite element analysis using ATENA was carried out on all reference and retrofitted specimens. The FEM models were validated with experimental outcomes. Subsequently, the validated models were utilized to develop a new simplified method for upgrading based on the advantages of RT1 and RT2. In the new proposed innovative Retrofitting Technique 3 (RT3), HHDP was replaced by Frictional-Bending Damper Plate (HFBDP) which dissipates energy based on friction and bending. The effectiveness and reliability of the proposed RT3 was investigated through a numerical analysis. The results of simulation showed that RT3 could efficiently achieve the intention of seismic retrofitting too.

At the end, as confirmed through experimental and numerical investigation, it is claimed that the all acceptance criteria of ACI Committee 374 [ACI 374.1-05] were effectively satisfied by the proposed retrofitting techniques.

Keywords: beam-column joint; retrofitting; seismic; analysis; design; energy dissipation; plastic hinge; inelastic deformation; corbel; friction

Kurzzusammenfassung

Die Arbeit enthält eine kategorisierte Übersicht von Nachrüst- und Verstärkungsmethoden bewehrter Balken-Stützen-Verbindungen aus der Literatur. Es zeigt sich, dass sowohl baulicher Eingriff, praktische Ausführung, Duktilität und Dauerhaftigkeit bezüglich seitlichen Widerstands, als auch ökonomische Randbedingungen die herausforderndsten Aspekte seismischer Verstärkungen gefährdeter Balken-Stützen-Verbindungen aus Stahlbeton sind.

Die seismischen Konstruktionsprinzipien von Stahlbetonrahmenkonstruktionen wurden entsprechend der Strategie für Nachrüstungen nach dem „capacity design concept“ untersucht. Dabei wurde der „beam sidesway mechanism“ für seismische Verstärkungen durch eine Verlagerung des plastischen Gelenks in geeigneter Entfernung zur Rahmenecke neu definiert. Danach werden durch Einführung innovativer Energiedissipationsgeräte, wie Multifunktionskonsole (HMFC) und Harmonika-Dämpfer-Platte (HHDP), innovative Verstärkungstechniken 1 und 2 (RT1 und RT2) vorgeschlagen. Die innovativen Geräte HMFC und HHDP als passives Energiedissipationssystem absorbieren Energie durch unelastische Verformung. Zur effizienten und ausgedehnten Bewertung und Anordnung erwarteter Widerstandshierarchie in Balken-Stützen-Verbindungen vor und nach der Verstärkung, wurde das Widerstands-Versagensfolge-Diagramm (SFSD) mit veränderter Ordinate vorgeschlagen. Zur Anwendung der eingeführten RT1 undRT2 und zum Erreichen der gewünschten Widerstandshierarchie wurde ein kompletter Entwurfsprozess präsentiert. Um das Verhalten und die Leistungsfähigkeit des vorgeschlagenen innovativen Geräts und Techniken zu untermauern, wurden umfassende numerische Analysen mit der nichtlinearen FE-Software ATENA durchgeführt.

Die vorgeschlagenen Verstärkungstechniken wurden experimentell mittels einer Serie von 5 Balken-Stützen-Verbindungen im ¾-Maßstab evaluiert, wobei zwei Einheiten als Referenz ohne Verstärkung und drei mit Verstärkung getestet wurden. Es wurde eine spezielle Belastungseinrichtung im Labor konstruiert und hergestellt, so dass die Prüfstücke auf dem Boden stehen und seitliche zyklische Last, mit der Maßgabe einer vertikalen statischen Last, an den Proben angreifen. An allen Proben wurde eine extrem harte Belastungsgeschichte weggesteuert eingetragen, die in Übereinstimmung mit [ACI 374.1-05] aus progressiv ansteigenden Verschiebungsamplituden besteht, wobei drei Zyklen (Druck und Zug) auf

einem bestimmten Driftniveau liegen. Durch die experimentellen Untersuchungen bestätigt sich die exzellente Leistungsfähigkeit der verstärkten Probestücke sowie die Annahme, dass RT1 und RT2 in der Lage sind das Widerstandsvermögen mit einem Minimum an Festigkeits- und Steifigkeitsverlusten beizubehalten. Wie erwartet stieg die Kapazität zur Energiedissipation drastisch an und der „beam sidesway mechanism“ bildete sich tatsächlich aus.

Letztlich wurde die nichtlineare FE-Analyse durch Benutzung von ATENA alle verstärkten und nicht verstärkten Proben angewendet. Das FE-Modell wurde durch die experimentellen Ergebnisse validiert. Anschließend wurden die validierten Modelle benutzt, um eine neue vereinfachte Methode zur Verbesserung zu entwickeln, die auf den Vorzügen von RT1 und RT2 basieren. In der neu vorgeschlagenen innovativen Verstärkungstechnik 3 (RT3) wurde das HHDP durch eine Biegereibungsdämpferplatte (HFBDP) ersetzt, welche Energie basierend auf Reibung und Biegung dissipiert. Die Effektivität und Funktionsfähigkeit der vorgeschlagenen RT3 wurde mit Hilfe numerischer Analysen untersucht. Die Ergebnisse der Simulation zeigten, dass RT3 die Intention seismischer Verstärkung ebenfalls effizient erzielen könnte.

Letztlich, wie durch experimentelle und numerische Untersuchungen bestätigt, wird behauptet, dass alle geforderten Kriterien des ACI-Komitees [ACI 374.1-05] durch die vorgeschlagenen Verstärkungstechniken befriedigt wurden.

Schlagworte: Balken-Stützen-Verbindungen; Nachrüstung; Seismisch; Analyse; Konstruktion; Energiedissipation; Plastisches Gelenke; unelastische Verformung; Konsole; Reibung

Acknowledgments

First, and foremost, praises and thanks to the God, the Almighty, for providing me the blessings to complete this research successfully.

I wish to express my deep and sincere gratitude and appreciation to my research supervisors Prof. Dr.-Ing. Steffen Marx, Prof. Dr.-Ing. Nabil A. Fouad, and Prof. Dr.-Ing. Jürgen Grünberg for giving me the opportunity to develop this research and for providing me the guidance, encouragement and supporting throughout the course of this study that made this project possible.

I would like to appreciate Dr.-Ing. Michael Hansen for his supports during experimental study.

I would like to thank the all colleagues of Institute of Concrete Construction (IFMA), Leibniz University of Hannover, particularly Mrs. Kerstin Bensch, Mrs. Simone Matern and my dear friend Dipl.-Ing. Jens Piehler for their kindly assistance.

All experiments were conducted at structural laboratory of Institute of Concrete Construction (IFMA), Leibniz University of Hannover. I would like to show gratitude to technical staffs of the laboratory Mr. Ernst Heine and Mr. Olaf Menze for their distinguished assistance during the experimental study. I wish also to show appreciation to the other institutes' staff in the structural laboratory, Mr. Lothar Beer, Mr. Karl-Heinz Hentschel, Dipl. Ing. Christian Fricke, Mr. Viktor Wall, and Mr. Gerd Hargesheimer for their advices and assistance.

Materials (FRP composites, adhesives and SAS670 bolts) donated by FIDIA, Hardwire, Sika and Stahlwerk Annahute are thankfully acknowledged.

Last but not least, I wish to express my deep thankfulness to my family for their patience to wait in loneliness for the right time. Thanks to my lovely sons Amirhossein and Amirreza for their accompanying. My sincere gratitude to my wife Sonia for her supports, encourages and patience during this study so that without her this work could not have been possible.

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Notation

A_{ch} cross-sectional area of a structural member measured to the outside edges of transverse reinforcement, mm^2

A_e effective joint cross-sectional area, mm^2

A_g gross area of concrete, mm^2

A_h the area of the third cycle to the drift ratio of 3.5%

A_j effective joint cross-sectional area, mm^2, computed from joint depth (h_c) times effective joint width (the overall width of the column, except where a beam frames into a wider column, effective joint width shall not exceed the smaller of: a) beam width plus joint depth, b) twice the smaller perpendicular distance from longitudinal axis of beam to column side

A_{s1} area of the beam top reinforcement, mm^2

A_{s2} area of the beam bottom reinforcement, mm^2

A_{sh} total cross-sectional area of transverse reinforcement including crossties within spacing s and perpendicular to dimension b_c, mm^2

$A_{sv,i}$ total area of the intermediate bars placed in the relevant column faces between corners of the column including bars contributing to the longitudinal reinforcement of columns, mm^2

A_{Tjh} total area of the horizontal hoops in a beam-column joint, mm^2

b_b width of the longitudinal beam, mm

b_c width of the column, mm

b_j effective joint width, mm, should not exceed the smallest of ($\frac{b_b+b_c}{2}, b_b + \frac{\sum mh_c}{2}, b_c$), where beam-column eccentricity exceeds $b_c/8$, m=0.3, otherwise m=0.5

b_{jj} effective joint width, if $b_c > b_w$: $b_{jj} = \min\{b_c; (b_w + 0.5h_c)\}$; if $b_c < b_w$: $b_{jj} = \min\{b_w; (b_c + 0.5h_c)\}$

b_w width of beam web, mm

d_b nominal diameter of bar, mm

E_0 initial elastic modulus for concrete, MPa

E_c secant elastic modulus at the peak stress for concrete, MPa

E_1 the peak lateral resistance for the positive lateral loading direction

E_2	the peak lateral resistance for the negative lateral loading direction
f_{cd}	design value of concrete compressive strength, MPa
F_{ps}	post-tensioning force of the bottom corbel, N
$f_c^{'}$	specified compressive strength of concrete, MPa
$f_c^{'ef}$	concrete effective compressive strength, MPa
$f_t^{'ef}$	the effective tensile strength, MPa
f_{cd}	design value of concrete compressive strength, MPa
f_{ck}	strength of concrete, MPa
f_{ctd}	design value of the tensile strength of concrete, MPa
f_{ctm}	mean value of tensile strength of concrete, given as $0.3f_c^{'(0.667)}$
f_y	specified yield strength of reinforcement, MPa
f_{yd}	design value of yield strength, MPa
f_{yt}	specified yield strength of transverse reinforcement, MPa
f_{ywd}	design value of the yield strength of the transverse reinforcement, MPa
h_c	overall cross-sectional depth of column, mm
h_{jc}	distance between extreme layers of column reinforcement, mm
h_{jw}	distance between the top and the bottom reinforcement of the beam, mm
h_v	vertical distance of horizontal LVDTs at the end of beam or HMFC, mm
h_h	horizontal distance of vertical LVDTs at the end of column, mm
h_x	Max. center-to-center spacing of crosstie legs on all faces of the column, mm
J_2	second invariant of stress deviator tensor
k	shape parameter the relation of Stress-strain for concrete
l_c	story height, length of column, measured center-to-center of the top and bottom beams, mm
l_{cn}	clear length of the column, mm
l_b	span length of beam, measured center-to-center of the column, mm
l_d	development length in tension of deformed bar based on the building codes, mm
l_{dh}	development length in tension of deformed bar with a standard hook, measured from critical section to outside end of hook, mm
l_{nb}	clear length of beam from face of columns, mm

M_{bc}	Joint moment at the beam joint interface, N.m
$\bar{M}_{bc}$	joint moment capacity of the exterior beam-column joint at the beam joint interface, N.m
M_{nbc}	beam bending moment capacity or beam yielding at joint interface, N.m
M_{ncb}	column bending moment capacity or column yielding at joint interface, N.m
N	column axial load, N
N_{Ed}	design axial force from the analysis for the seismic design (the minimum value from load combination), is assumed positive when compressive, N
s	center-to-center spacing of transverse reinforcement within the joint, mm
r_{ec}	reduction factor of the compressive strength
r_{et}	reduction factor of the tensile strength
V_b	shear force across the beam, N
V_c	shear force in the column above the joint, from the analysis in the seismic design situation, N
V_{col}	shear force in the column above the joint, N
$\bar{V}_{col}$	story shear capacity of the as-built exterior beam-column jointcorresponding to the certain strength, N
$\bar{\bar{V}}_{col}$	story shear capacity of the retrofitted exterior beam-column jointcorresponding to the certain strength, N
v_d	normalized design axial force of column
V_{jhd}	horizontal shear force acting on the concrete core of the exterior joint, N
V_{jh}	horizontal shear force acting on the concrete core of the exterior joint, N
V_{nb}	beam shear strength, N
V_{nc}	column shear strength, N
w	the crack opening, mm
w_c	crack opening at the complete release of stress, mm
x	normalized strain
z_b	internal moment arm in the beam, mm
z_{ps}	proper moment arm of the bottom corbel post-tensioning, mm
α	stress multiplier for beam longitudinal bars
β	relative energy dissipation ratio
γ	joint shear strain
γ_{Rd}	model uncertainty factor for the design value of resistance for beam

	longitudinal bars, given as 1.2
γ_{xz}	shear strain
ε	normal strain
ε_c	strain at the peak stress $f_c'^{ef}$
ε_x	normal strain of joint panel in the x direction
ε_z	normal strain of joint panel in the z direction
ε_φ	strain in joint panel in an arbitrary direction (diagonal) with an angle of φ measured counter clockwise from the x axis
Θ'_1	drift ratio in positive direction
Θ'_2	drift ratio in negative direction
σ_c^{ef}	concrete compressive stress for the relation of Stress-strain for concrete, MPa
η	reduction factor on concrete compressive strength due to tensile strain in transverse direction
$\sum M_{nc}$	sum of nominal flexural strength of columns framing into the joint, N.mm
$\sum M_{nb}$	sum of nominal flexural strength of beams framing into the joint, N.mm

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
BD	Bond Deficiency
CFRP	Carbon Fiber-Reinforced Polymer
CORDIS	Community Research and Development Information Service
CSA	Canadian Concrete Design Code
DIN	Deutsches Institut für Normung
DTAM	Digital World Tectonic Activity Map
FRP	Fiber-Reinforced Polymer
EN	Europäisiche Norm
HFBDP	Hayatrouhi Frictional-Bending Damper Plate
HHDP	Hayatrouhi Harmonica Damper Plate
HMFC	Hayatrouhi Multi Functional Corbel

HPFRC	High-Performance Fiber-Reinforced Concrete
GFRP	Glass Fiber-Reinforced Polymer
GLD	Gravity Load Designed
GSHAP	Global Seismic Hazard Assessment Program
LVDT	Linear Variable Distance Transducer
NASA	National Aeronautics and Space Administration
NPO	Non Profit Organisationen
NSM	Near-Surface-Mounted
RC	Reinforced Concrete
RT1	Retrofitting Technique 1
RT2	Retrofitting Technique 2
RT3	Retrofitting Technique 3
SD	Shear Deficiency
SRP	Steel Fiber-Reinforced Polymer
SFSD	Strength and Failure Sequence Diagram
UNIDO	United Nations Industrial Development Organization
USGS	Unite States Geological Survey