Seismic Fragility And Post-Earthquake Reparability of Concrete Frame With Low-Bond High-Strength Rebar Reinforced Concrete Column

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Abstract

To further study the global seismic behaviour and post-earthquake reparability of RC building frames with the proposed self-centring columns with low-bond high-strength reinforcements (LBHSRs), incremental dynamic analysis (IDA) of five-floor and ten-floor frame archetypes under excitation by twenty ground motions (GMs) was performed. First, the pushover results indicated that the use of LBHSR could substantially improve the yield and ultimate lateral drift of both the archetypes, although the archetype had a smaller longitudinal reinforcement ratio (LR) of the LBHSR and similar seismic resistance. The dynamic response results indicated that the archetype with LBHSRs exhibited a smaller residual story lateral drift although the effectiveness of the use of LBHSR to reduce seismic response was not apparent for the archetype subjected to a low-intensity earthquake. The seismic fragility results showed that LBHSR was more effective for preventing seismic collapse than for attaining the immediate occupancy (IO), life safety (LS), and collapse prevention (CP) limit states. Furthermore, the higher the LR, the lower was the likelihood of seismic collapse. The fragility curves of the residual story lateral drifts indicate that the use of LBHSR can significantly mitigate the residual deformation in the DS1, DS2, and DS3 damage states. Moreover, the effectiveness increases with the increase in the LR and earthquake intensity. Comparisons of residual story lateral drifts between the predicted results and IDA results indicated that the present calculation models are not suitable for predicting residual deformation. The model needs to be studied further.

Keywords: seismic fragility; post-earthquake reparability, LBHSR, IDA, residual deformation.

40 **1 Introduction**

41 As is well-established, earthquakes are one of the major natural disasters encountered by humans. The
42 severe earthquakes of recent decades have caused substantial human casualties and socio-economic losses.
43 Furthermore, the indirect severe damage caused by an earthquake to the local politics, economy, human
44 psychology, and social development are considerable (Wang and Zhao 2018). Earthquake-resistant
45 structural engineering has been developing over many generations. The countries highly vulnerable to
46 earthquakes have proposed many performance-based design methods of engineering structures, e.g., the
47 three-level seismic design method proposed in the Chinese code (GB50011-2010, 2010) and two-stage
48 seismic design method proposed in the Japanese code (AIJ, 2016).

49 The design concept for preventing engineering structures from collapsing to protect human lives is
50 effective. Human casualties in the recent earthquakes in Japan were significantly less than those in earlier
51 earthquakes. However, these design methods result in significant seismic damage of engineering
52 structures in the form of plastic hinge. Engineering structures that sustain severe seismic damage incur
53 substantial post-earthquake repair expense. Occasionally, a structure that does not undergo seismic
54 collapse needs to be demolished because of the substantial repair cost. This substantially hinders the
55 rehabilitation of society and human life because the repair of infrastructure including buildings and
56 bridges is highly time-consuming and requires substantial socio-economic resources. Therefore, scholars
57 have indicated that engineering structures should satisfy the traditional design objective of ensuring
58 human safety as well as limit the seismic damage and residual deformation to aid the repair of the
59 structure after an earthquake (Kawashima 1997). Furthermore, the residual deformation after an
60 earthquake also poses a risk to structural safety during post-earthquake fires (Wang et al. 2020).
61 “Resilient City” should become the next research emphasis for seismic-resistant engineering structures.

62 Since 1963, Housner (1963) determined that structures did not sustain severe seismic damages in the
63 1960 Chile earthquake because the connection between the main structure and foundation had been

weakened. Since the completion of his theoretical analysis of the working mechanism of rocking structures, the design principle of weakening the connection of a structure to reduce the overall structural seismic damage has been receiving increasing attention of scholars worldwide. Since the 1990s, American and Japanese scholars have carried out substantial research on the seismic performance of rocking structures. To control the rocking amplitude of a rocking structure, the post-tensioned (PT) prestressing technique was used to provide the self-centring force for the structure. This type of rocking structure is called self-centring (SC) structure. The research on SC structures has focused mainly on precast reinforced concrete (RC) shear walls and RC frames. Prof. Richard Sause of Lehigh University and Prof. Yahya C. Kurama of the University of Notre Dame carried out extensive and comprehensive works. Their early test results demonstrated that the residual displacement and seismic damage of an SC shear wall structure are marginal and that the force–displacement curve exhibits a flag-shape, which reveals a strong self-reset capability. However, its energy dissipation capacity is low (Kurama et al. 1999). The energy dissipation capacity could be improved significantly by additional dampers. The simulation analysis showed that the application of rocking or SC structural elements was favourable to the seismic resistance of an entire structure (Zibaei and Mokari 2014; Kurama and Shen 2008). Based on this consideration, Panian et al. (2008) have used SC shear walls in the construction of David Brower Center. Priestley et al. (1996) attempted to use partially debonded post-tensioned prestressed steel bars in precast reinforced-concrete beam–column joints to improve seismic performance.

Kawashima et al. (1997) comprehensively investigated the residual deformation of single-degree-of-freedom (SDOF) structures. It was observed that the residual deformation of structures is controlled mainly by the ductility ratio, post-yield stiffness, and yield stiffness ratio. Priestley and Christopoulos et al. (Christopoulos et al. 2003; Pampanin et al. 2008) theoretically analysed the residual deformation of SDOF and multi-degree-of-freedom (MDOF) structures. According to the study results of Kawashima and Priestley et al., a series of formulas were proposed in the Seismic Design Code for Bridge

88 Engineering of Japan (JRA 2017) and American FEMA specification (FEMA 2012).

89 Meanwhile, although rocking and SC structures have been developed for nearly 30 years, these have
90 not been applied extensively in practical engineering. Certain problems prevent its popularisation and
91 development: 1) Economic cost (MacRae 2013; Roke et al. 2013). The PT technique increases the
92 difficulty of structural design. In addition, the anchorage construction of pre-stressing bars in shear walls
93 and frames is difficult. These increase the construction cost. However, owing to the uncertainty of
94 earthquake, the economic advantages afforded by the high seismic performance of rocking and SC
95 structures in the life cycle may be offset by the increased cost involved in the preliminary design and
96 construction of the structure. This substantially reduces the willingness of residents to apply it. 2) Design
97 difficulty. As mentioned earlier, the working mechanism of rocking and SC structures differs from those
98 of traditional structures. This implies that a new design method is required, which increases the design
99 challenge for engineers. 3) Inherent deficiency of the PT technique (Wang and Zhao 2018). Loss of pre-
100 stress occurs in the life cycle of SC structures. It is difficult to evaluate the influence of pre-stressing loss
101 on the seismic performance of SC structures. Consequently, it is worthwhile to develop SC structures
102 with simple and inexpensive technology (Wang and Zhao 2018).

103 The authors have theoretically analysed the influence of the bond strength of longitudinal
104 reinforcement on the seismic behaviours of RC columns. The results indicated that a decrease in the bond
105 strength can substantially improve the ductility of RC columns and that the yield process can be delayed
106 such that the longitudinal reinforcement can be maintained in the elastic region (Wang and Sun 2020a;
107 Wang and Sun 2020b). Based on the above theoretical foundation, the authors proposed the use of low-
108 bond high-strength reinforcement (LBHSR) as the tensile longitudinal rebar of RC columns.

109 Certain experimental programs were conducted by the authors to investigate the seismic behaviours of
110 the proposed RC columns. The details of the structural forms of the proposed test RC columns with
111 LBHSRs are shown Fig.1. As is evident from the figure, the structural forms of the proposed RC columns

are identical to those of conventional RC columns. This substantially reduces the difficulties of design and construction and further reduces the cost of this new type of SC building structures.

The low-bond property of longitudinal rebars can be realized by two methods: 1) use partially debonded normal high-strength twisted reinforcement (PDHSR) (see Fig.1 (a)); and 2) use low-bond-strength high-strength reinforcement (LBHSR) (see Fig.1 (b)). PDHSR can be realized by wrapping the surface of high-bond high-strength reinforcement (HBHSR) with a poly vinyl chloride (PVC) sleeve or steel tube, as shown in Fig.1 (a). A type of LBHSR is shown in Fig.2. This type of high-strength reinforcement (called SBPDN 1423/1275 (hereinafter referred to as SBPDN) rebar) was developed by a Japanese steel company in the 1950s. Thereafter, it was gradually adopted in the civil engineering field. For example, it has been used in the construction of the high-speed railway of China and is used as transverse reinforcement in Japan. The types of LBHSR with different bond strengths have been standardized (GB/T 5223.3-2017), as shown in Fig.2. The surface of an SBPDN rebar displays ribs (similar to USD 685 rebar) rather than spiral grooves. According to the test results of Funato et al. (2013), the bond strength of an SBPDN rebar was a fifth of that of conventional high-strength USD 685 rebars with ribs.

A tie plate is required to prevent bond-slip at the middle and both ends of RC columns with LBHSRs. The longitudinal reinforcement can be anchored by the tie plates at both top and bottom of the column or through 90–180° hooks. The debonded length and bond strength can be freely selected by the designer according to the demands of structural performance. To further reduce the seismic damage and prevent premature shear failure of an RC column, the column section can be confined by a fibre polymer plastic (FRP) jacket and steel tube (ST), as shown in Fig.1. Further details of the proposed RC columns with LBHSRs are available in the companion papers (Wang et al. 2019; Wang et al. 2020). The experimental results indicated that the RC columns with LBHSRs displayed strong SC capacity. In addition, the residual deformation after seismic loading was smaller compared with that of RC columns with normal

136 bond-strength reinforcements. Furthermore, the seismic damage of the proposed RC columns was
137 marginal (see Fig.3). This further reduced the difficulty and cost of post-earthquake repair.

138 The seismic behaviours of beam–column joint assembly reinforced by either PDHSRs or normal steel
139 rebars were experimentally investigated (Wang 2021). The results indicated that the use of PDHSR in the
140 columns was favourable to the improvement of the ductility of beam–column joints. Meanwhile, the use
141 of both beams and columns with PDHSRs can substantially improve the seismic behaviours of beam–
142 column joints, including their seismic resistance, ductility, and residual deformation. This prevents the
143 deterioration of the seismic behaviours of the RC frame with the proposed columns because of the use of
144 PDHSR.

145 The seismic behaviours of RC columns with LBSHSRs or PDHSRs subjected to lateral cyclic loading
146 have been investigated extensively and comprehensively through experiment and evaluation (Wang et al.
147 2019; Wang et al. 2020; Wang et al. 2021). Further study is required to better comprehend the influence
148 of the proposed SC columns with LBHSRs on the seismic responses of an entire RC frame. This paper
149 presents the time-history analysis results of five-floor and ten-floor building frame archetypes with
150 concrete columns reinforced by steel rebars with different bond properties. Nonlinear numerical models
151 considering steel-bond slip were proposed in OpenSees (2018), and incremental dynamic analysis (IDA)
152 was performed to obtain seismic responses of the designed building frame archetypes based on twenty
153 earthquake ground motions. The results were used to determine 1) seismic fragility curves for evaluating
154 seismic collapse 2) and residual drift curves for estimating the post-earthquake reparability of the
155 designed building archetypes. Furthermore, the residual story lateral drifts obtained from the IDA were
156 evaluated using the equations proposed in the FEMA and JRA codes. The works of this study provide a
157 better understanding of the proposed RC column application for designing an inexpensive SC–RC frame
158 by using simple techniques.

159

2 Numerical models of concrete frame with LBHSR reinforced columns

2.1 Design of building frame archetypes

Five-floor and ten-floor RC building frames were designed to represent the expected design variation of short and medium-rise buildings. Fig. 4 shows the plan view and elevation of the building frames. Each floor contains many moment resisting frames (MRFs) in both horizontal and vertical directions. There are four and seven bays in the vertical and horizontal directions, respectively. It is assumed that the building frames has identical story height and bay width of 3.2 m and 4.0 m, respectively. The intermediate MRF was selected to perform the time-history analysis. The columns of the first floor were assumed to fix the foundation. It was assumed that the width of the slabs was 4.0 m, and the membrane element was used to model the slabs. Live loads of 2.0 kN/m² were assigned to all the building floors. The dead load was calculated according to the self-weights of all the members including the columns, beams, and slabs.

The building archetypes were assumed to be located in Dongchuan District, Kunming City, Yunnan Province of China. It is in the southwestern of China and is one of the areas that are highly vulnerable to earthquake. The latitudes and longitudes are 102° 47'–103° 18' and 25° 57'–26° 32', respectively. According to the seismic design code of China (GB 50011-2010), the building archetypes satisfy the Seismic design category (SDC) 9, which is the highest seismic vulnerability. The maximum considered earthquake (MCE) spectral acceleration S_a was 0.4g. The soil was assumed to be of Class II. The natural eigenperiod T_g is 0.55 s. The first modes of the building archetypes were considered in the nonlinear time-history analysis. The fundamental periods T_1 of the five-floor and ten-floor archetypes were 0.5 s and 1.0 s, respectively. These were calculated by Eq. (1) proposed in the ASCE 7 code (ASCE/SEI 7-10, 2010):

$$T_1 = 0.1N \quad (1)$$

where T_1 = fundamental period and N = number of stories of the building frame archetype.

184 The seismic design results are shown in Table 1. The design earthquake accelerations of the building
185 archetypes were calculated from their fundamental periods using the equations for calculating earthquake
186 acceleration proposed in the seismic design code of China (GB 50011-2010).

187 The compressive strength of the concrete was assumed to be 36.0 MPa, and the confinement ratio of
188 the column section was assumed to be 1.22. Meanwhile, the beam section did not consider the
189 confinement to concrete. The dimensions of the column and beam are shown in Fig.4 (b). The columns
190 had a square cross-section with dimensions of 500 × 500 mm. Meanwhile, the size of the beam section
191 was 400 × 450 mm. Twelve longitudinal rebars with a diameter of 25 mm were assigned to the beam
192 sections to yield a longitudinal rebar ratio of 3.27%. The yield strength of the longitudinal reinforcement
193 used in the beams was assumed to be 454 MPa.

194 Table 1 shows the basic information of the designed column longitudinal reinforcement. Two types of
195 longitudinal reinforcements were used in the columns: yield strength of 500 MPa to represent high-bond
196 normal-strength reinforcement (HBNSR), yield strength of 942 MPa with a debonding length of 960 mm
197 (60% of the shear span of the column), or bond strength of 3.0 MPa to represent LBHSR. Sixteen
198 HBNSRs or LBHSRs with a diameter of 25 mm was designed to reinforce the column section to yield a
199 longitudinal reinforcement ratio (LR) of 3.14%. However, owing to the higher yield strength of LBHSR,
200 the seismic base force resistances of the building archetypes with LBHSRs was higher than those for the
201 archetypes with HBNSRs with identical LR (see Fig.10). To eliminate the effect of the yield strength of
202 LBHSR and further study the influence of the bond property of longitudinal reinforcement on the seismic
203 responses of building archetypes, 12 D22 and 4 D12 LBHSRs (to yield a lower LR of 2.09%) was
204 assumed to reinforce the columns of another building archetype scenario for providing a seismic base
205 force resistance similar to that of the building archetype scenario with 16 D25 HBNSRs (see Table 1).
206 The damping ratio of both the archetypes was assumed to be 5.0%.

207

2.2 Ground motions

Twenty ground motions (GMs) including ten far-field (FF), five near-field pulse (NFP), and five near-field no-pulse (NFNP) GMs summarized by FEMA P695 (FEMA 2009) were selected to perform the nonlinear time-history analysis. The acceleration data of these GMs were downloaded from ‘PEER Strong Ground Motion Database’. The basic information including locations, years, magnitude, peak ground acceleration (PGA), and duration are shown in Table 2. The corresponding acceleration spectral with a damping ratio of 5.0% is shown in Fig.5. The Chinese seismic design code (GB 50011-2010) includes four SDCs: Classes 6, 7, 8, and 9. The curve of the average value of all the selected GMs and the design earthquake acceleration spectral of the four SDCs are compared in Fig.5. As is evident from the figure, the maximum earthquake acceleration of the average curve is between the curves of SDCs 7 and 8. Furthermore, the curve is close to the SDC 8 curve when the period is larger than T_g ($= 0.55$ s).

2.3 Nonlinear finite element model

The two-dimensional OpenSees element model of the MRF is shown in Fig.6. The beam and column were simulated by the NonlinearBeamColumn element. Fiber-Section was used to simulate the beam and column sections. The layout and mechanical properties of the longitudinal reinforcements were identical to those of the design illustrated in Fig. 4 (b). The concrete and steel models were UniaxialMaterial Concrete 02 with linear tension softening and Steel 02 (Giuffré–Menegotto–Pinto model) with isotropic strain hardening, respectively. A zeroLength shear spring element was used to simulate the shear behaviour of the beam–column joint (Alath and Kunnath 1995) (see Fig.6). Meanwhile, the steel-bond slip was simulated through the simplified elastic modulus of longitudinal rebars of beam and column. The mechanical property of the shear spring was calculated by the Pinching 4 material model. The model of shear strength versus shear strain proposed by Kim and LaFave (Kim et al. 2008; Kim et al. 2009; Kim and LaFave 2007) was used to determine the parameters of the Pinching 4 material model.

2.4 Considerations of column sectional confinement and steel-bond slip of LBHSR

The confined concrete compressive strength was used to simulate the FRP-confined column section. The confinement ratio was calculated using Eq. (2) of the model proposed by Lam and Teng (2003). Meanwhile, Eq. (3) proposed by Sun and Sakino (1996) was used to calculate the confinement ratio owing to ST. It was verified that the above two material models could be used to reasonably simulate the confinement effect of FRP and ST on the test column sections (Wang et al. 2019; Wang et al. 2021).

$$K = \frac{f_{cc}}{f_{c0}} = 1 + 3.3 \frac{f_l}{f_{c0}}; \quad f_l = \frac{2f_{frp}t}{d} \quad (2)$$

$$K = 1 + 4.1 \left(\frac{2}{D_t / t_s - 2} \right) \cdot \frac{f_{yt}}{f_{c0}} \quad (3)$$

where K = confinement ratio of concrete, f_{cc} = confined concrete strength, f_{c0} = unconfined concrete strength, f_l = confining pressure owing to FRP, f_{frp} = tensile strength of FRP, t = total thickness of FRP, t_s = thickness of ST, and d = diameter of confined concrete. D_t is the diameter of the section considering the thickness of the steel tube, and f_{yt} is the yield strength of the hoops and steel tubes.

The authors used a modified Young's modulus of steel for the simulation to estimate the steel-bond slip of the LBHSR of the columns. The applicability of this method has been validated (Wang et al. 2019; Wang et al. 2021). The calculation model of the steel-bond slip of LBHSR is shown in Fig.7. As is evident, the steel-bond slip for PDHSR contains two parts: one is from the debonding region, and the other is caused by the bond region of the longitudinal reinforcement. Meanwhile, the steel-bond slip for LBHSR is caused only by the bond region. The calculation of the steel-bond slip of longitudinal reinforcement with different bond strengths was based on the equations proposed by Sezen et al. (Sezen and Selzler 2008; Sezen and Chowdhury 2009; Lodhi and Sezen 2012). According to the anchorage length of the longitudinal reinforcement, two scenarios were considered (see Fig.7). Scenario (a): the

255 anchorage length is adequate. Scenario (b): the development length of the yield longitudinal
 256 reinforcement l_{dy} exceeds the anchorage length L_{emd} . For simplicity, unified equations are proposed for
 257 estimating the steel-bond slip:

$$258 \quad E_{sm} = \begin{cases} \frac{4uE_s L_{emd}}{4uL_{emd} + 4ul_{pd} + f_y d_b}; & \text{Scenario } a \\ \frac{f_y}{\varepsilon_y + 0.5(\varepsilon_e + \varepsilon_y) \left(1 - \frac{l_{pd}}{L_{emd}}\right)}; & \text{Scenario } b \end{cases} \quad (4)$$

$$259 \quad \varepsilon_e = \varepsilon_y + \frac{4u(L_{emd} - l_{pd})}{f_y d_b} \quad (5)$$

260

261 where d_b = diameter of reinforcing bar, ε_e = axial strain at the end of longitudinal rebar, ε_y = axial yield
 262 strain of reinforcing bar, f_y = yield stress of reinforcing bar, and u = bond strength between concrete and
 263 longitudinal rebar. L_{emd} = embedment length of longitudinal rebar, E_{sm} = modified Young's modulus of
 264 steel, E_s = Young's modulus of steel, and l_{pd} = debonding length of steel rebar.

265

266 **2.5 Validation of numerical model**

267 To validate the applicability of the above OpenSees models, the same OpenSees element model was
 268 used to simulate the experimental seismic responses of the columns with normal high-strength
 269 reinforcements and LBHSRs tested by the authors (Wang et al. 2019; Wang et al. 2021). The calculation
 270 of the confinement of FRP and ST and the simulation of steel-bond slip shown in Eqs. (2)–(5) were also
 271 used in the simulations of test columns. The OpenSees models for cantilever and double-curvature beam–
 272 columns are shown in Fig.8. The comparisons of the predicted and experimentally determined seismic
 273 responses of the test columns are shown in Fig.9. As is evident from the figure, the proposed OpenSees
 274 element models shown in Fig.8 can be used to predict the seismic behaviours (including the stiffness,
 275 seismic resistance, ductility, and residual lateral drift) of test columns reinforced by longitudinal

reinforcements (PDHSRs, LBSHSRs, HBHSRs) with good accuracy. As indicated, the experimentally determined early stiffness in the negative direction of the specimen columns PPD0, PPD4, and PPD6 differed marginally from the predicted results. This was attributed to the seismic damage in the negative direction initiated when the test column was thrust in the positive direction. It should be noted that the experimentally determined seismic behaviours of the test columns FATSB and FATUS in the negative direction did not agree with the predicted results. This was attributed to the fact that the capacity of the hydraulic jack during the testing was smaller than the seismic resistance capacity of these two test columns. The maximum lateral drift cycles were up to 3.5% and 2.0% for FATSB and FATUS, respectively (Wang et al. 2021).

285

3. Static pushover analysis

An inverted triangular load pattern following the Chinese seismic design code GB 50011-2010 (2010) was used in the pushover analysis. The comparisons of the pushover results of the archetypes with HBNSRs and LBHSRs are shown in Fig. 10. It is evident from the figure that both five-archetype and 10-archetype with an identical LR (= 3.14%) of LBHSRs displayed better seismic responses (including higher seismic resistance and higher deformability) compared with the archetypes with HBNSRs. The lateral drifts at the peak lateral forces of the two archetypes with LBHSRs were 180% and 241%, respectively, higher than those of the archetypes with HBNSRs. When the LR of LBHSRs was decreased from 3.14% to 2.09%, the seismic base shear forces of the building archetypes were similar to those of the archetype reinforced with HBNSRs having an LR of 3.14%. However, the peak lateral drifts of the five-floor and ten-floor archetypes with LBHSRs were increased by 139% and 222%, respectively, compared with the archetypes with HBNSRs. For the two LR of LBHSR (3.14% and 2.09%), both the archetypes had reduced stiffness compared with that for HBNSRs (LR = 3.14%) from a certain lateral drift level, moreover, lower the LR, lower was the stiffness (see Fig.10). These observation were consistent with the

experimental results shown in the references (Wang et al. 2019; Wang et al. 2021). This can be attributed to the fact that the low-bond property of LBHSR delayed the development of the yielding process of the longitudinal reinforcement. Thereby, the archetypes with LBHSRs had larger yield roof lateral drifts, $R_{y,eff}$ (see Table 1). When the LR of LBHSR was 3.14%, the $R_{y,eff}$ of the archetypes with LBHSRs was 2.81 and 1.88 times, respectively, that of the archetypes with HBNSRs. Meanwhile, because the LR was 2.09%, the $R_{y,eff}$ values were still increased by 48.6% and 3.3%, respectively, for the five-floor and ten-floor archetypes.

The over-strength factor Ω and period-based ductility μ_T were used to quantify the seismic behaviour of the building frame archetypes (see Table 1). The average yield displacement of the calculated results of three methods including first yield, the geometric graphic method, and the equivalent elastoplastic energy method defined by Park (1989) was used in this study to calculate the period-based ductility. Meanwhile, the point of 20% strength loss (0.8V) was considered as the ultimate displacement. As shown in Table 1, the Ω s of both five-floor and ten-floor archetypes were similar when the LRs of HBNSR and LBHSR were 3.14% and 2.09%, respectively. Consequently, the use of LBHSR can reduce the amount of longitudinal reinforcement. However, when the LR of LBHSR was 3.14%, the Ω s were larger by 31% and 16%, respectively, compared with the archetypes with HBNSRs. As shown in Table 1, the μ_T of the five-floor archetypes with LBHSRs was marginally smaller compared with that attained with HBNSRs. Meanwhile, the ten-floor archetypes with LBHSRs had larger μ_T compared with the ten-floor archetypes with HBNSRs.

319

320 4. Dynamic response and seismic fragility evaluation

Fig.11 shows the dynamic responses including the history-time responses of the maximum story lateral drifts and hysteretic behaviours of the five-floor archetypes with HBNSRs and LBHSRs subjected to GM 12 and GM13. The amplitude-scaling factors (CFs) of GM 12 and GM 13 were 1.0. As is evident from

Fig.11 (a), the maximum story lateral drift of the archetype with HBNSRs ($LR = 3.14\%$) subjected to GM 12 was approximately 2.0% , whereas the value for the archetype with LBHSRs was higher than 2.0% for LR s of 3.14% and 2.09% . However, for the high-intensity earthquake GM 13, the maximum story lateral drift of the archetype with HBNSRs ($LR = 3.14\%$) was approximately 10.0% . It could be assumed to be a collapse. However, the values of the archetypes with LBHSRs were decreased to 6.42% and 8.23% for LR s of 3.14% and 2.09% , respectively. As shown in Fig.11 (b), when subjected to GM12, the base shear force of the archetype with LBHSRs was smaller than that of the archetype with HBNSRs. For GM 13, the archetype with LBHSRs having an LR of 3.14% displayed larger base shear force, whereas the base shear forces of the archetypes with HBNSRs having an LR of 3.14% and those with LBHSRs having an LR of 2.09% were equal. Furthermore, the stiffness of the archetype with LBHSRs was also smaller than that of the archetype with HBNSRs when the lateral drift entered the large deformation stage. These observations were in agreement with the static pushover results shown in Fig.10. It was also observed that the residual lateral drifts of the archetypes with three longitudinal reinforcement scenarios were similar when subjected to GM12. Meanwhile, the archetypes with LBHSRs having LR s of 3.14% and 2.09% displayed smaller residual lateral drifts compared with the archetypes with HBNSRs ($LR = 3.14\%$) subjected to GM13. Furthermore, larger the LR of LBHSR, smaller was the residual lateral drift (see Fig.11(a)). The dynamic responses shown in Fig.11 indicate that the only apparent effect of the use of LBHSR is the reduction in the seismic dynamic responses of structures subjected to a high-intensity earthquake.

The lateral drift responses in conjunction with the height of the archetype are shown in Fig.12. Identical to the observations in Fig.11, the maximum story lateral drift response of the archetype subjected to the relatively moderate or low-intensity earthquake G12 was significantly smaller compared with that for GM 13. However, regardless of the earthquake intensity, LBHSR was remarkably effective in reducing the residual story lateral drift, as shown in Fig.12 (b). Similar conclusions were obtained from

348 comparisons of the dynamic responses of the ten-floor archetypes reinforced by LBHSRs and HBNSRs.

349 IDA was performed to capture the entire seismic response of the designed building frame archetype.
 350 The time-history analysis was conducted repeatedly through amplitude-scaling of each GM until the
 351 archetypes were assumed as the failure. The time-history analysis was terminated when the inter-story
 352 drift ratio exceeded 10%. Fig.13 shows the relationships between the earthquake intensities at period T_1
 353 and maximum story lateral drift R_p . Based on the IDA results, the seismic fragilities of the archetypes
 354 were estimated by a lognormal cumulative distribution function (Vossoghi and Saiidi 2012; Muntasir and
 355 Alam 2014; Baker 2015), shown in Eqs. (6) and (7). Furthermore, the logarithmic standard deviation of
 356 $S_a(T_1)$ was calculated by Eq. (6).

$$357 \quad P = [C / S_a(T_1) = x] = \Phi\left(\frac{\ln(x / \mu)}{\beta}\right) \quad (6)$$

$$358 \quad \beta = \sqrt{\frac{1}{n-1} \sum_{i=1}^n \left(\ln\left(\frac{S_a(T_1)}{\mu}\right) \right)^2} \quad (7)$$

359 where $P [C / S_a(T_1) = x]$ = probability that a ground motion with $S_a(T_1)$ of x would cause the
 360 structure to collapse, $\Phi()$ = standard normal cumulative distribution function, μ = median of the fragility
 361 function (the $S_a(T_1)$ level with 50% probability of collapse), β = standard deviation of LN $S_a(T_1)$, and n
 362 = the selected amount of GMs.

363

364 The FEMA 356 guideline (FEMA 2000) defines four limit-states to elaborate the seismic damage
 365 status of building structures: immediate occupancy (IO), life safety (LS), collapse prevention (CP), and
 366 collapse. IO, LS, and CP are assigned when the maximum lateral inter-story drifts of 1.0%, 2.0%, and
 367 4.0%, respectively. FEMA P-58-1 (FEMA 2009) defines collapse as one of the following scenarios: side-
 368 sway failure (lateral dynamic instability) characterized by loss of lateral stiffness and development of P-
 369 delta instability, loss of vertical-load carrying capacity of gravity or seismic-force-resisting components

owing to earthquake-induced drifts, and exceedance of non-simulated failure criteria consisting of force or deformation limits beyond which components cannot reliably resist the load. For simplicity, the maximum lateral inter-story drift of 10.0% was assumed as the collapse limit state (Tarfan et al. 2019). This rule was also adopted in this study.

Fig.14 shows the seismic fragility probabilities of the exceedances of the IO, LS, CP, and collapse limit states of both five-floor and ten-floor archetypes. As indicated, for the IO and LS limit states, the possibility curves of the five-floor archetypes with LBHSRs were larger than those of the five-floor archetypes with HBNSRs. Moreover, the median collapse capacity S_{CT} representing half of the GMs caused the limit state of the five-floor archetypes with HBNSRs to be the largest. Comparing the CP and collapse limit states, it was observed that the possibility of the archetype with HBNSRs was the highest. This implies that the use of LBHSR could reduce the likelihood of CP and seismic collapse limit states. Furthermore, larger the LR, lower is the likelihood. The above statements indicate that the use of LBHSR had no apparent effect on the resistance to the marginal seismic limit states such as IO and LS. However, it was favourable to the CP and seismic collapse limit states. Furthermore, higher the LR of LBHSR, lower is the likelihood of seismic collapse. A similar conclusion can be obtained by observing the possibility curves of the ten-floor archetypes. Comparing the probability curves of the IO, LS, and CP limit states, the possibility curves of the ten-floor archetype with LBHSRs (LR = 3.14% and 2.09%) were larger than that of the ten-floor archetype with HBNSRs having an LR of 3.14%. However, the possibility curves of the seismic collapse limit state of the ten-floor archetypes with LBHSRs (LR = 3.14% and 2.09%) were significantly smaller compared with the HBNSR scenario. Moreover, the advantage of using LBHSR was more apparent for the IO, LS, and CP limit states of the five-floor archetype than for those of the ten-floor archetype. However, a comparison of the difference in the possibility curves of the archetypes with HBNSRs and LBHSRs revealed that the application of LBHSR in the ten-floor archetype was more effective for resisting the collapse limit state. The collapse margin ratio (*CMR*) is used to

represent seismic safety and is defined as the ratio of the S_{CT} to the 5%-damped spectral acceleration of the MCE GMs at the fundamental period, $S_a(T_1)$ (FEMA 2009). The *CMRs* of all the building archetypes with different longitudinal reinforcement scenarios at four SDCs of Chinese seismic code (GB 50011-2010) are shown in Table 1. As is evident, although the seismic resistance of the archetype with LBHSRs having a lower LR of 2.09% was similar to that of the archetype with HBNSRs having an LR of 3.14%, the *CMRs* of the former at each SDC were increased by 6.7% and 13.75% compared with the latter for the five-floor archetype and ten-floor archetype, respectively. Furthermore, when the LR of LBHSR was increased from 2.09% to 3.14%, the increase ratio was enhanced further to 26.51% and 33.75% for the five-floor archetype and ten-floor archetype, respectively. Hence, it was concluded that the global seismic response of an RC building frame is substantially affected by the use of LBHSR. The effect of LBHSR is more apparent when the building frame experiences a large deformation level such as the collapse limit state.

406

407 **5. Post-earthquake reparability and evaluation of residual story lateral drift of archetype**

408 **5.1 Analysis of fragility of residual story lateral drift**

Residual deformation has been used as an index to estimate the post-earthquake damage of buildings, bridges, and other infrastructure. FEMA P 58-1 (FEMA 2012) defined four damage states (DSi) ranging from the onset of damage of nonstructural components to near-collapse of the structure. The residual story drift ratios Δ / h of DS1, DS2, and DS3 are defined as 0.2%, 0.5%, and 1.0%, respectively. Meanwhile, the Δ / h of the DS4 damage state for limited, moderate, and high ductility systems are 1.0%, 2.0%, and 4.0%, respectively. DS1 represents the state wherein no structural realignment is necessary for structural stability, and the adjustment and repair of only certain nonstructural and mechanical components that are sensitive to elevator rails, curtain walls, and doors are required. DS2 represents a realignment of the structural frame and related structures are need to be maintained, limited degradation

in structural stability is permitted to occur in nonstructural and mechanical components. DS3 represents the scenario where significant structural realignment is required to restore the margin-of-safety for lateral stability. Furthermore, the repair of the structure is not economically or practically feasible. DS4 represents the scenario wherein the structure is in danger of collapse after an earthquake owing to an excessive residual drift.

The relationships between earthquake intensity and the residual story lateral drift of archetypes with HBNSRs and LBHSRs are shown in Fig.15. As is evident from the figure, the increase in the stiffness of the fragility curves of the archetypes with LBHSRs was larger than that for the archetypes with HBHSRs. This was particularly so for identical LR. Moreover, the number of residual story lateral drifts higher than 2.0% of the archetype with LBHSRs having an LR of 3.14% is not adequate for a residual drift fragility analysis similar to the one for the other two longitudinal reinforcement scenarios. As shown in Fig.10, the pushover curves of both archetypes indicate that the building frame could be assumed to be a moderate ductility system. Consequently, only the DS1, DS2, and DS3 damage states of the archetypes were compared in this study.

The comparisons of the possibility curves of three damage states of the archetypes with HBNSRs and LBHSRs are shown in Fig.16. As is evident from the figure, regardless of the damage states, both five-floor and ten-floor archetypes with LBHSRs generally exhibited significantly smaller probability compared with that for the archetypes with HBNSRs. Although the archetypes with HBNSRs having an LR of 3.14% and LBHSRs having an LR of 2.09% had similar seismic resistance (see Fig.10), the possibility curve of the archetype with LBHSRs having an LR of 2.09% was smaller than that of the archetype with HBNSRs having an LR of 3.14%. Moreover, when the LR of LBHSR was increased from 2.09% to 3.14%, the possibility curve of the archetype was decreased further, as shown in Fig.16. This implies that the low-bond property of longitudinal reinforcement influences the reduction in the residual deformation of building frames. A closer inspection reveals that the LBHSR is more effective for

mitigating the residual drift for the low-rise five-floor building archetype. Furthermore, the possibility curves of archetypes are similar at marginal earthquake intensity regardless of the type and LR of longitudinal reinforcement. In addition, the difference in the possibility curves is increased with the increase in the earthquake intensity. It can be concluded that the use of LBHSR is highly effective for reducing the residual deformation of an RC building frame. Furthermore, it is more effective when the building is subjected to a large earthquake.

5.2 Evaluation of residual story lateral drift

The previous study determined that the residual deformation of structures is significantly related to the ratio of post-yield stiffness to yield stiffness. However, it is not dependent on the natural fundamental period, soil condition, or ductility (Tarfan et al. 2019). Japan Road Association (JRA) (JRA 2017) and Federal Emergency Management Agency (FEMA) of United States (FEMA 2012) proposed certain equations for evaluating the residual deformation of bridge and building structures based on the study results of Kawashima et al. (Tarfan et al. 2019; JRA 2017) and Christopoulos et al. (Christopoulos et al. 2003; Pampanin et al. 2008). Therefore, the residual story lateral drift of the building archetype can be estimated using the proposed equations in the JRA and FEMA codes.

The equations to calculate the residual drift of structure proposed by FEMA P58-1 (FEMA 2012) is as follows:

$$R_{rd} = \begin{cases} 0 & R_p \leq R_y \\ 0.3(R_p - R_y) & R_y \leq R_p \leq 4R_y \\ (R_p - 3R_y) & R_p \geq 4R_y \end{cases} \quad (8)$$

where R_{rd} = residual lateral drift, R_p = transient peak lateral drift, and R_y = yield story lateral drift.

Kawashima (1997) extensively studied the residual drift of engineering structures subjected to an

464 earthquake. They proposed the following equation, which has been adopted by the JRA code (JRA 2017):

465
$$R_{rd} = C_R (\mu_r - 1)(1 - r) R_y \quad (9)$$

466 where R_{rd} = residual lateral drift, C_R = correction coefficient (= 0.6), μ_r = response ductility, r = ratio of
467 the post-yielding stiffness to initial yield stiffness, and R_y = yield story lateral drift.

468

469 As shown in Eqs. (8) and (9), the residual lateral drift is determined by the yield story lateral drift R_y
470 and coefficient r . To obtain these two factors, the relationships between the lateral story shear force and
471 the lateral drift of the collapse story of the archetypes were determined according to the static pushover
472 results. Subsequently, R_y and r were calculated. Furthermore, the average of the results of the three
473 methods (namely, first yield, the geometric graphic method, and the equivalent elastoplastic energy
474 method proposed by Park) was adopted as R_y (Park 1989).

475 Fig.17 shows comparisons between the predicted (using the FEMA and JRA codes) and IDA results of
476 the residual story lateral drifts of both the archetypes. As indicated, the residual story lateral drifts of both
477 the archetypes with different types and LR of longitudinal reinforcements were generally overestimated.
478 The average values of the ratios of predicted results to IDA results (Pred./IDA) are 2.61 and 4.11 for the
479 FEMA and JRA codes, respectively. The standard deviations of the FEMA and JRA codes are 6.13 and
480 9.52, respectively, whereas the maximum Pred./IDA of the FEMA and JRA codes are 62.02 and 98.19,
481 respectively. The results indicate that the prediction using the FEMA code was generally better than that
482 using the JRA code. The comparison results imply that the calculation models proposed by the current
483 provisions are not capable of predicting the residual story lateral drifts of RC building frames with good
484 accuracy. It is necessary to conduct investigations to further refine the equation for predicting residual
485 deformation.

486

6 Conclusions

The authors proposed the use of low-bond high-strength reinforcement (LBHSR) as longitudinal rebar to develop a self-centring RC column. This study further investigated the seismic fragility and post-earthquake reparability of RC building frames with the proposed columns based on the IDA results. A smaller longitudinal reinforcement ratio of LBHSR was assumed for designing the similar seismic resistance of the archetype to investigate the influence of the low-bond property on the seismic response. In addition, the longitudinal reinforcement ratio of LBHSR and HBNSR was assumed to be identical to study the effect of longitudinal reinforcement ratio. The results of this study could be summarized as follows:

1. Notwithstanding similar seismic base shear force resistance, the use of LBHSR with a lower LR can improve the yield roof lateral drift and ultimate lateral drift (at 0.8 V) of both low-rise and mid-rise building frame archetypes owing to the low-bond property. Furthermore, higher the longitudinal reinforcement ratio (LR), better are the results.
2. The stiffness of the archetype with LBHSRs was decreased from a certain lateral drift because of the bond-slip of the LBHSR. Furthermore, an increase in the LR could increase the stiffness.
3. The dynamic response results indicated that the LBHSR was highly effective for reducing the seismic lateral drift responses only for the structure subjected to a high-intensity earthquake. However, its use effectively reduced the residual story lateral drift regardless of the earthquake intensity.
4. The seismic fragility curves indicate that LBHSR is more effective for preventing seismic collapse as well as for attaining a higher LR and lower collapse probability. However, the advantage is limited for the IO, LS, and CP limit states.
5. The fragility curves of the residual story lateral drifts of both the archetypes show that the LBHSR can substantially reduce the exceedance probability of the damage states DS1, DS2, and DS3.

Furthermore, the effectiveness is increased with the increase in the LR and earthquake intensity.

6. The calculation models for predicting the residual lateral drift proposed by the present JRA and FEMA provisions are not capable of assessing the residual drifts with good accuracy. Therefore, further study is required.

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Compliance with ethical standards

Conflict of interest: The authors declare that there is no conflict of interest regarding the publication of this paper.

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Table 1. Basic information of building archetypes

Archetype		Five-floor			Ten-floor		
Column	Type	HBNSR	LBHSR		HBNSR	LBHSR	
longitudinal rebar	Number	16-D25	12-D22+ 4-D12	16-D25	16-D25	12-D22+ 4-D12	16-D25
	LR (%)	3.14	2.09	3.14	3.14	2.09	3.14
Height (<i>m</i>)		16.0			32.0		
T_1 (<i>s</i>)		0.5			1.0		
$S_{a, MCE}$ (<i>g</i>)		0.4			0.2336		
V_m (<i>kN</i>)		1168.0			1140.4		
Ω		1.24	1.26	1.62	1.23	1.27	1.43
$R_{y, eff}$ (%)		0.74	1.10	2.08	0.41	0.71	0.77
μ_T		10.76	6.61	9.03	6.42	6.72	7.65
<i>CMR</i>	SDC Six	74.50	79.50	94.25	40.00	45.50	53.50
	SDC Seven	24.83	26.50	31.41	13.33	15.17	17.83
	SDC Eight	12.42	13.25	15.71	6.67	7.58	8.92
	SDC Nine	9.31	9.94	11.78	5.00	5.69	6.69

Note: D = diameter of longitudinal reinforcement; LR = longitudinal rebar ratio; T_1 = fundamental period; $S_{a, MCE}$ = maximum considered earthquake spectral acceleration; V_m = base shear force calculated according to the designed earthquake spectral acceleration; Ω = Over-strength factor; $R_{y, eff}$ = effective yield roof lateral drift ratio; μ_T = period-based ductility; CMR = collapse margin ratio.

Table 2. Information of earthquake motions for nonlinear time-history analysis

No.	Earthquake					Recording Station Name	PGA (g)	DT (s)	Duration Time (s)
	RSN.	Mag.	Year	Name	R_{rup} (km)				
Far-field (FF)									
1	953	6.7	1994	Northridge	17.15	Beverly Hills-Mulhol	0.15	0.01	20.00
2	960	6.7	1994	Northridge	12.40	Canyon Country-WLC	0.48	0.01	20.00
3	1602	7.1	1999	Duzce, Turkey	12.04	Bolu	0.74	0.01	55.90
4	1787	7.1	1999	Hector Mine	11.70	Hector	0.34	0.01	45.31
5	1111	6.9	1995	Kobe, Japan	7.10	Nishi-Akashi	0.51	0.01	40.96
6	1116	6.9	1995	Kobe, Japan	19.15	Shin-Osaka	0.23	0.01	40.96
7	900	7.3	1992	Landers	23.60	Yermo Fire Station	0.24	0.02	44.00
8	848	7.3	1992	Landers	19.74	Coolwater	0.28	0.02	44.00
9	1633	7.4	1990	Manjil, Iran	12.55	Abbar	0.52	0.02	53.52
10	725	6.5	1987	Superstition Hills	11.20	Poe Road (temp)	0.45	0.01	22.30
Near-field: Pulse Records Subset (NFP)									
11	723	6.5	1987	Superstition Hills-02	3.50	Parachute Test Site	0.42	0.01	22.35
12	828	7.0	1992	Cape Mendocino	8.18	Petrolia	0.59	0.02	36.00
13	1063	6.7	1994	Northridge-01	6.50	Rinaldi Receiving Sta	0.87	0.01	20.00
14	1086	6.7	1994	Northridge-01	5.30	Sylmar - Olive View	0.73	0.02	40.00
15	1605	7.1	1999	Duzce, Turkey	6.58	Duzce	0.40	0.005	26.00
Near-field: No Pulse Records Subset (NFNP)									
16	126	6.8	1976	Gazli, USSR	5.50	Karakyr	0.71	0.0066	13.52
17	160	6.5	1979	Imperial Valley-06	2.66	Bonds Corner	0.60	0.01	51.60
18	165	6.5	1979	Imperial Valley-06	8.40	Chihuahua	0.28	0.01	51.60
19	825	7.0	1992	Cape Mendocino	6.96	Cape Mendocino	1.49	0.02	30.00
20	1048	6.7	1994	Northridge-01	12.10	Northridge - Saticoy	0.42	0.01	30.00

Fig. 1. Reinforcement details of the proposed RC columns (units: mm): (a) with PDHSRs; (b) with LBSHSRs

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(a) SBPDN 1275/1420 rebar

(b) USD 685 rebar

Fig.2. High-strength reinforcement with different bond strength levels

Fig. 3. Typical seismic damages of test columns with LBHSRs: (a) RC columns with PDHSRs; (b) RC columns with LBSHSRs

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(a)

(b)

Fig.4. Building frame archetypes: (a) plain view; (b) elevation

Fig.5. Acceleration spectral of ground motions

Fig.6. OpenSees element model of building frame archetype

Fig. 7. Steel-bond slip considering debonding and bond effects

Fig.8. OpenSees element model for column: (a) cantilever; (b) double-curvature

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Fig. 9. Typical hysteretic curves of RC columns with LBHSRs: (a) PDHSR; (b) LBSHSR

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Fig.10. Pushover results of archetypes

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(a)

(b)

Fig.11. Dynamic responses: (a) maximum story lateral drift history-time responses; (b) hysteretic behaviors

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Fig.12. Lateral drift responses along with the floor: (a) maximum story lateral drift; (b) residual story lateral drift

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Fig.13. Relationships of earthquake intensity versus peak roof lateral drift: (a) five-floor archetype; (b) ten-floor archetype

Fig.14. Seismic collapse probability of archetypes: (a) five-floor archetype; (b) ten-floor archetype

Fig.15. Relationships of earthquake intensity versus peak roof lateral drift: (a) five-floor archetype; (b) ten-floor archetype

Fig.16. Seismic collapse probability of archetypes: (a) five-floor archetype; (b) ten-floor archetype

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Fig.17. Comparisons of residual drifts of archetypes between prediction and IDA result data

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