INFLUENCE OF CORROSION ON FAILURE MECHANISMS AND LIFETIME SEISMIC RISK ASSESSMENT OF LOW-DUCTILITY RC FRAMES

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# ABSTRACT

Passive control systems, such as buckling-restrained braces (BRBs),

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# INTRODUCTION

**Earthquakes** are one of the major catastrophic events that render serious damage to the buildings and infrastructure systems. Across the globe, old reinforced concrete buildings have experienced substantial damage, and in some cases collapse, from seismic events. The primary cause of building damage, of about 88%, is due to ground shaking which is the major reason for huge economic and human losses and this is directly dependent on the exposure conditions of the area [1–4]. For example, the economic loss was about **120 Billion USD** in Japan due to the 1995 Kobe earthquake [4]. Such a high magnitude of losses can prove detrimental to the economy of the nation as it requires large funds and time to restore to the pre-seismic conditions. This magnitude of the economic and human loss number can be higher if the **infrastructure systems** and buildings are **not designed for seismic forces**. In addition to the seismic risk, there exists several other deterioration phenomena when the structures are exposed to harsh environmental conditions – this includes sulphate attack, carbonation, corrosion, *etc.* Corrosion in the RC structures is the most widely observed phenomena that has a substantial effect on service life of the structure [5].Recognizing this fact, a study in the domain of seismic hazard assessment of ageing infrastructure like bridges has been done by various researchers – as these are vital aspects economically and strategically for the nation – while similar research work for building structures are still at its infancy [6–9]. Hence, seismic vulnerability assessment for RC buildings is important as they have exposure to high fatality numbers during an earthquake event. Along with exposure to the high number of human lives, RC buildings also host important business offices, critical agencies, and manufacturing units; and the failure of RC buildings can have an immediate effect on the economy and daily life of the people.

RC buildings, can experience **corrosion** which causes deterioration of the structure over a long period which in turn reduces the life of the structure [10,11]. Corroded reinforced concrete (RC) structures are highly susceptible to extreme damage conditions compared to the new pristine structure (or recently constructed) during a seismic event; this is aggravated with low ductility which is designed as per the traditional codes (non-seismic design) [6,12,13]. When located in an aggressive environment, the seismic risk of the **existing RC building** increases with the level of corrosion which invariably reduces the service life of the building structure [14,15]. Hence, exposure of such RC building to the future seismic event makes it vulnerable with the combined effect of non-seismic design and deterioration effect due to corrosion. Thus, it is necessary to carry out vulnerability checks to access the danger due to future hazardous events so that acceptable repair and retrofit measures can be done on this existing structure.

Corrosion of rebars in an RC member (especially columns of building) reduces the effective cross-sectional area of the rebars which invariably affects the strength and performance of the structure under lateral load.Corrosionin longitudinal reinforcement and transverse reinforcement have different deterioration effects on the failure mechanism. Experimental test carried on corroded columns based on percentage mass loss only in **longitudinal reinforcement** shows that there is a significant reduction in the flexural strength of columns, reduction in energy dissipation and drift capacity [16–18]. As **transverse reinforcement** has a higher probability of being corroded before the longitudinal reinforcement due to the smaller distance to the corrosive atmosphere, there is a significant change in the behavior of the column having corrosion in both reinforcements. This gap was studied by Vu and Li (2018) [19,20] and it was found that the 25% corrosion level in transverse reinforcement (stirrups) can significantly reduce the lateral load capacity and, in addition, change the failure mechanism from flexure to **flexure-shear failure**. It was also found, in the same study, that under high axial load columns with high corrosion faced sudden **axial failure**. Thus, modelling of **shear properties** becomes imperative for studying the effect of corrosion on the shear capacity of the RC members. These modelling techniques for shear are discussed in various research papers [21–23] which accounts for the **flexure-shear interaction of columns** during the application of lateral or cyclic load on the columns.

Until now major research was done in **single-column** tested under axial load with cantilever action or double curvature. Few experimental research works were conducted to study the seismic performance of corroded **moment**-**resisting frames** considering the effect of corrosion. Karapetrou et al. (2016) [24] provided a detailed study on low, mid and high rise 2-dimensional frames by considering probabilistic modelling of corrosion for assessing the seismic performance using increment dynamic analysis (IDA). Modelling of 2D non-linear beam-column frames was done in OpenSees [25] with fiber sections with a fixed base and soil-structure interaction (SSI). It was found that by incorporating the SSI parameters will hugely affect the expected seismic fragility of the frames. The study, done by Couto et al. (2020) [26], carried out the sensitivity analysis of corrosion rate on the vulnerability assessment of RC building frames (three types – low, mid and high rise) in Portugal. Non-linearity in fiber section modelling and rebar area reduction was considered for studying the corrosion effect on these ageing frames while doing the vulnerability assessment. It was found that the seismic vulnerability increases by 20% due to corrosion. Another research by Di Sarno and Pugliese (2020) [27] had conducted a numerical evaluation of RC building frames for a different level of exposure, different field motions and degradation with smooth rebars. A new approach of defining material in the finite element modelling (FEM) was discussed in the paper to account for the degradation effects and surrogate modelling was used to capture maximum shear. It was shown that the seismic vulnerability increases by 39% to 63% at 25 years of age depending on the type of field motions adopted during the assessment. Afsar Disaj et al. 2021 [28] on the other hand plotted the fragility curves considering the combined effects of naturally occurring sequential main shock – aftershock (MS-AS) and progressive deterioration due to corrosion for single bay RC frame. This study incorporated the effect of bond-slip phenomena based on the model by Zhoa and Sritharan (2007) [29]. It was concluded that the seismic fragility is slightly higher for MS-AS analysis compared to MS analysis. These studies on RC frames do not comment on the shear failure that occurs in old structures which **are low in ductility due to non-seismic design**. Research work on single columns expounds on the change in the behavior in failure from flexure to flexure-shear under the influence of corrosion as discussed earlier. There arises a need to study the failure mechanism of existing RC frames due to seismic loading with low ductility which is vulnerable to shear failure. Hence, this study tries to fill the gap by modelling low ductile frames which predominantly fail under shear during a seismic event. Also, in the present study, non-linear time history analysis (NLTHA) and time-dependent fragility curves for corroded and non-corroded frames will be carried out to study the effect of the earthquake on the low ductile frame.

# FRAMEWORK FOR LIFETIME SEISMIC VULNERIBILITY ASSESSMENT

# CASE STUDY STRUCTURE: DETERIORATION AND FINITE ELEMENT MODELLING

From the post-earthquake investigations carried out by several researchers point out, critically, the strong beam/weak column joint present in the structure as the common failure mechanism. This type of failure is independent of the geometric and mechanical properties of the building which are predominantly not designed for the seismic loads [23,30]. The older version of the codes did not have sufficient seismic detailing which causes the designed frames/joints to be low in ductility. The building designed pre-1989 (ACI times) did not consider seismic forces, thus the columns are primarily designed for axial loading. But columns are important members to transfer the high lateral loads due to horizontal movement of the earthquake action which determines the seismic performance of the RC building [31].

Corrosion has been observed to be the most significant and primary environmental degradation factor (including corrosion due to chloride ingress and carbonation of concrete surface) which adversely affect building’s mechanical property, especially the strength over a long period which is considered as the ageing effect [32,33]. Corrosion of an RC frame causes reduction of ductility, loss of rebar area, breakage of the bond between rebar and the surrounding concrete, spalling of concrete cover which eventually leads to loss of cover and the rebar is exposed directly to the atmosphere. This reduces the noteworthy load-carrying capacity of the RC frame against the lateral load which leads to a precarious brittle failure mechanism [12,34,35].

This premature degradation of RC buildings due to corrosion is a major concern for safety against earthquakes and subsequently the negative environmental impact it has due to its material consumption in retrofit and repairs. This premature deterioration of RC structure due to corrosion only was found to be 70-90% of the investigated cases [33]. It was estimated that a total value of $2.5 trillion (USD) is the cost of corrosion globally, and this is equivalent to 3.4% of the global GDP. However, for some countries, it can be as high as 5% of their annual GDP [36]. This when combined with the seismic event, the corrosion cost can be much higher and, in addition, cause human losses. Thus, the present study considers a three-storey three-bay RC moment resisting frame which is a low ductile frame (*viz*. non-seismically designed) and low-rise building [23]. The frame is considered to be in the California region where it is exposed to significant seismic activity and corrosion due to coastal conditions.

## Description of Case-Study Frame

The case study frame, adopted from Freddi et al. (2021) [23], has a total height of 10.75m (35.3 feet) with an interstorey height of 3.66 m (12 feet) and a constant bay width equal to 5.49 m (18 feet). The frame is designed as per the non-seismic guidelines from the ACI 318-89 [37] and without any seismic detailing provisions is considered in the frame. The column sizes are 300 x 300 mm and the beam sizes are 230 x 460 mm at each floor. The concrete compressive strength, =24 MPa (cube strength), and the reinforcing bar of steel Grade 40 with a yield strength of = 276 MPa. The other details of the case study frame can be found in Figure 3.1 which are adopted from Bracci et al. [38] and Aycardi et al. [39],.. The FE model strategy is followed as per the Freddi et al., [40,41] which helps in monitoring brittle local failure mechanism which is important for low-ductile frames.

## Time-Dependent Deterioration Modeling of Case-Study Frame

## Finite Element Modelling for As-built and Deteriorated Frame

For the modelling of the two-dimensional model of the case study frame, the finite element (FE) package OpenSees [25] is used. The schematic representation for the case-study frame is shown in Figure XX giving the size of the frame components and reinforcement details. In addition, the modelling features of the OpenSees model also include the plastic hinge locations, shear and axial springs and interior and exterior **beam-column** joints. For simulating the non-linear flexural hysteretic response of columns and beams and capturing the shear failure for the low ductile frame, *nonLinearBeamColumn* and *beamWithHinges* elements are used, respectively. The center part of the beam *beamWithHinges* consists of an elastic element, while for the two plastic hinge regions at the element end and the full length *nonLinearBeamColumn* element fiber section is adopted [42]. The **fiber section** consists of unconfined and confined concrete definition (cover and core concrete respectively) defined with non-linear degrading *Concrete02* material model and the longitudinal reinforcement is defined as layers using the *Hysteretic* **material** model. The confinement factor is considered for the core concrete using the model given by Mander et al. (1988) [43] to consider the enhancement of core concrete strength. For obtaining analytical results that closely represent the experimental results the damage and pinching parameters, and unloading stiffness are calibrated manually which is discussed in the next section. For modelling the slab, as per ACI 318-89, unconfined concrete material model is adopted with with an effective width of four times the beam’s width. The rigid-floor diaphragm is modelled by assigning high axial stiffness to the beams. Gravity loads are assigned as uniformly distributed loads (UDL) on the beams and the masses are assigned as concentrated at the beam–column junction.

A *ZeroLength* element **(spring) for shear and axial** is used at top of each column by assigning them the *LimitState* uniaxial material in OpenSees [25]. To axial failure is monitored using the limitCurve *Axial* and only triggered when the limit is reached as discussed in Fabio et al. (2021) [23]. The shear limit curve (*limitCurve Shear* – in OpenSees) is adopted based on the model given by Elwood (2004) [44] which is derived empirically. This model can be applied for only shear failure columns [23], the frame is joint is combined with ASCE-41 [45] proposed force-controlled shear limit surface. This is incorporated in OpenSees using ***LimitState* uniaxial material model** and *limitCurveShear* model which helps capture the failure when the shear limit or drift limit curve is reached. The shear limit in terms of shear strength is calculated by the model proposed by Sezen and Moehle (2004) [46]. These, *LimitState* material model and *limitcurveShear* model properties include the degrading slope of the total response, , which is computed as per the proposed shear-friction model discussed in Baradaran Shoraka et al. (2013) [47]. The total response is calculated by computing the value of the unloading stiffness () and the slope of the degrading curve () [44]. To avoid the convergence issues in the analysis, the final residual shear strength () is assumed to be equal to 20% of the initial shear strength ().

~~As shear failure is accompanied by cracking in the RC member~~**~~, bond-slip phenomena~~** ~~in the longitudinal reinforcement are required to be captured while performing the cyclical test. The longitudinal bar slip causes additional strain in the fiber section which is given by the bilinear stress-slip relationship established by Ghannoum and Moehle (2012) [48].~~ *~~zeroLengthSection~~* ~~is used to incorporate the bar slip in the lateral load direction by multiplying the slip factor to the strain values in the~~ *~~uniaxialMaterial~~* ~~(concrete material) in OpenSees~~.

To model the **low-ductility joints** for the RC frame, a two-node *zeroLength* rotational joint spring and four rigid offsets is used as illustrated in the work done by Jeon et al. (2015) [49]; and shown in Figure XX. In this model, beams and columns are continuous, while the joint model controls their relative rotation. Low ductility can arise due to insufficient traverse reinforcement and short embedded lengths of reinforcement within the beam-column joints. To account for the short anchorage length, a reduced shear strength is considered as done by Jeon et al. (2015) [49] as shown in Figure XX. Also, to account for the pinching behavior *Pinching4* [25] material model is used in the beam-column joint response as shown in Figure XX.

### Validation with past experimental studies

For the purpose of validating the OpenSees [25] modelling technique of simulating the shear-critical response, four square RC columns are analyzed under cyclical loading to validate the flexure-shear response (*flexures-shear interaction FSI)*. For the purpose of obtaining a good agreement with the experimental result, *zeroLengthSection* is used to model the bond-slip phenomena based on the relationship provided by Ghannoum and Moehle (2012) [48]. The columns for validation are selected based on the flexure-shear failure and includes uncorroded and corroded specimens. Table 3.1 gives the basic details and properties of the selected specimens used for the validation exercise. Specimen U2 and C5 are of similar properties but U2 is tested without any corrosion and C5 is tested with corrosion of rebars [20]. Specimen S1 is a pristine column with no corrosion while CC1 is a short column with corrosion which is expected to observe shear failure. All the columns information is taken from the experimental test research so help verify the modelling aspect adopted in this study.

~~Figure 3.1a and 3.1b are the pushover and shear limit plots for each specimen; from the plots, it can be concluded all the specimens observe flexure-shear failure. That is, lateral load capacity for each specimen intersects with the shear limit curve calculated as per by Sezen Moehle 2004.~~

The plots of Figure 3.1a and 3.1b are hysteric plots for all the column specimens modelled as per the description in the above section. Particularly from Figure 3.1a – specimen S1 – it can be illustrated that the column with low ductility will have a sharp decrease in lateral load capacity after reaching the shear limit. The hysteric plot for S1 is a good match validating our aim of modelling shear element and bar-slip section for the analysis. However, the hysteric plot of C5, CC2 and U2 does not have a good match with the experimental results. Also, the peak lateral load of the test is always higher for these three specimens and the initial stiffness upto the elastic range is clearly visible to be higher than that of the stiffness observed in the experimental curve. Part of the reason can be attributed to the experimental setup where the ends are not fully rigid and there is a presence of rigid body rotation at the fixed end. Another reason for the overprediction of the peak lateral strength would be the inelastic deformation in the reinforcement bars. However, the deterioration and pinching phenomena is well captured in all of the hysteric plots but the drawback remains in modelling which required manual inputs of these parameters to match with the experimental results. To summarize, the modelling discussed in the above section captures the flexure-shear failure in a satisfactory manner that can be extended to the frame under this study.

**Table 3.1 Database of the columns selected for the validation exercise**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Specimen | U2 | C5 | CC2 | S1 |
| Reference | Vu and Li (2018a) [20] | Vu and Li (2018a) [20] | Vu and Li (2018b) [19] | Sezen and Moehle (2006) [50] |
| Section, *b x h* (mm x mm) | 350 | 350 | 350 | 457 |
| Column length, *l* (mm) | 1780 | 1780 | 1080 | 2946 |
| Experiment test configuration | Double Curvature | Double Curvature | Double Curvature | Double Curvature |
| Shear span to depth ratio, (*a/d*) | 3.18 | 3.18 | 1.93 | 3.74 |
| Concrete strength (MPa) | 38.1 | 31.3 | 28.8 | 21.1 |
| Longitudinal reinforcement | 20φ – 8nos | 20φ – 8nos | 20φ – 8nos | 28.7φ – 8nos |
| Yield strength of long bar ( | 550 | 550 | 550 | 438 |
| Ultimate strength of long bar( | 635 | 635 | 635 | 645 |
| Transverse reinforcement | φ7.8@50 | φ7.8@50 | φ7.8@50 | φ8@305 |
| Yield strength of trans reinf. ( | 300 | 276.8 | 267.8 | 476 |
| Ultimate strength of trans reinf. ( | 465 | 465 | 465 | 645 |
| Nominal Cover, (mm) | 32.8 | 32.8 | 32.8 | 34.9 |
| Axial Load, *P* (kN) | 958.6 | 1165 | 352.8 | 667 |
| Corrosion level-transverse (%)\* | - | 15.5% | 21.4% | - |
| Corrosion level longitudinal (%)\* | - | 3.9% | 3.1% | - |
|  |  |  |  |  |
| Ductility ratio | 4.2 | 3.6 | 2.32 | 2.88 |
| Yield displacement, (mm) | 24.92 | 13.3 | 17.5 | 26 |

|  |  |  |  |
| --- | --- | --- | --- |
| **A** |  | **B** |  |
| **C** |  | **D** |  | |
| **Figure 3 – A) S1 B) U2 C) C5 D) CC2** | | | | |

~~Seismic importance~~

~~Experimental test of RC frames and discuss its results~~

~~Gap to be filled and motivation~~

* ~~Building deterioration versus the infra systems~~
* Recent studies on Building corrosion deterioration
* ~~Lacking in for RC building~~
* ~~Low ductile frames (cause of old of existing frames)~~

Key motivations – need to add more points

~~NON-ductile frame building; dollar loss; loss of due to non-ductile; close to marine conditions which aggravates corrosion. Several experimental of frames; ageing and corrosion may decrease life; recognizing this fact several experimental study;~~

~~Social impact of erthquake events~~

~~Seismic design evolution impact~~

6.2ht cm and 8.29 cm wt . Line width 1.5; Time new roman 16

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(Baradaran Shoraka et al. 2013)

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