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

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# ABSTRACT

**Keywords**: Corrosion; Reinforced concrete frames; Seismic retrofit; Fragility curves; Seismic risk variability.

# INTRODUCTION

One of the most common concerns influencing the structural performance of reinforced concrete (RC) buildings is the durability issues caused by exposure to harsh environmental conditions. RC buildings when located in harsh environmental conditions (coastal areas, industrial sites) result in ageing and deterioration of structural components. Corrosion in the RC structures is the most widely observed reason of deterioration that has a substantial effect on the service life of the structure [CITE Darmawan 2010]. Corrosion deterioration of RC structures leads to a significant reduction in the cross-sectional area of reinforcing steel along with alteration in mechanical properties of concrete and steel resulting in loss of load bearing capacity of sections, elements, and the whole structure. Past studies reveal that 70-90% of the deterioration in RC buildings is due to corrosion of RC components [CITE]. Moreover, corrosion deterioration is estimated to cost around $2.5 trillion (USD) globally which is comparable to 3.4% of global GDP. For some countries, it can be as high as 5% of their annual GDP [CITE Koch et al. 2016]. In addition to continuous exposure to environmental hazards, when located in moderate to high seismic regions, RC buildings are also exposed to intermittent seismic threats. Across the globe, older reinforced concrete buildings have experienced substantial damage, and in some cases collapse due to earthquake events. About 88% of building damage is due to ground shaking resulting in huge economic and human losses [CITE 1-4]. For instance, the 1976 Tangshan earthquake resulted in 255,000 casualties and 10 billion Yuan loss and the recent 2011 Tohoku earthquake resulted in 30,000 casualties and $235 billion loss [CITE]. Such a high magnitude of losses can prove detrimental to the economy of the nation as large funds and time are required to restore the building to the pre-seismic conditions. The magnitude of the economic and human losses can be higher when the civil infrastructure systems and buildings are designed with antiquated design standards and undergoing corrosion deterioration. Recognizing this fact, in the domain of seismic risk assessment of existing ageing infrastructure systems, several research has been carried out to assess the lifetime vulnerability of existing bridges under threats stemming from combined ageing and seismic hazards [CITE]. Such studies are deemed necessary as bridges are lifeline infrastructure systems, and must remain in service after an extreme event to provide essential public services during the post-hazard recovery phase. Similar research work for existing ageing building structures is also required, however, it is still at its infancy [CITE 5-6 papers]. Seismic risk assessment for existing RC buildings is necessary 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 community.

Recognizing the need for studying the influence of corrosion on lifetime seismic performance of existing RC buildings, recently few numerical studies are conducted on RC moment resisting frames [CITE 5-6 papers]. Pitilakis et al. (2014) [CITE] conducted a detailed study on low, mid and high rise moment resisting frames by considering probabilistic modelling of corrosion and assessed the lifetime seismic performance using time-dependent seismic fragility curves. Modelling of 2D frames was done in OpenSees [CITE] incorporating corrosion deterioration and soil-structure interaction (SSI) effects. It was found that time-dependent corrosion deterioration and SSI effects hugely influence the seismic fragility curves of the frames. Couto et al. (2020) [CITE] carried out sensitivity analysis of corrosion rate on the vulnerability assessment of older RC building frames built between 1960-1980 (three types – low, mid and high rise) in Portugal. For vulnerability assessment, time-dependent corrosion deterioration effects such as rebar area reduction and concrete strength reduction were considered in the analytical model. Non-linear static analysis based seismic fragility analysis results reveal that probability of failure of corroded frame increases by 20% due to corrosion. Additionally, results reveal that the concrete strength degradation had more influence than reduction of the rebar diameter in the seismic capacity. Di Sarno and Pugliese (2020) [CITE] conducted a numerical evaluation of RC building frames for different level of exposure and degradation conditions. The primary and secondary effects of corrosion deterioration of RC members such as reduction in rebar area and mechanical properties of steel and concrete were considered. The findings of the non-linear dynamic analyses revealed that corrosion damage and deterioration increased the roof and inter-storey drift-ratios, as well as a significant decay of the base shear capacity and early collapse were noted for high-levels of corrosion. Recently Dizaj et al. 2018 [CITE] assessed the influence of non-uniform corrosion on the seismic vulnerability of code conforming (ductile) and code nonconforming (non-ductile) RC frames using detailed non-linear finite element model. While the results highlight the influence of corrosion on seismic damage limit states and vulnerability, it was concluded that consideration of non-uniform corrosion spatial variability has insignificant effect on global nonlinear behavior and seismic vulnerability of corroded RC frames. Additionally, several researchers have also underlined the impact of considering corrosion deterioration in RC frame that may be subjected to multiple seismic shocks (main shocks, or, main shock-aftershock sequences) along the design life [CITE Di Sarno & Pugliese, 2020; Dizaj et al. 2021]. While the above-mentioned studies contributed significantly in the field of seismic risk assessment of ageing building structures, these studies do not comment on the non-ductile shear failure of RC columns that may emerge crucial for older non-seismically designed RC buildings undergoing corrosion deterioration, as discussed next.

Most of the existing buildings in seismic prone regions across the globe are non-ductile as they are built prior to the introduction of modern seismic design codes. Due to the limited ductility capacity, these buildings are significantly vulnerable under severe ground shakings. Corrosion deterioration of these older non-ductile buildings further reduces the lateral load carrying capacity. Identifying the negative influence of corrosion on the lateral load carrying capacity of buildings, several experimental campaigns have been conducted on RC columns under cyclic loading [CITE Meda et al. 2014, Goksu and Ilki 2016, Ma et al. 2012, Yang et al. 2012]. Results from these studies reveal that the flexural strength and ductility capacity of RC columns decreases significantly when the longitudinal reinforcement bars sustain higher levels of corrosion. Additionally, the energy dissipation capacity of RC columns also decreases significantly with the increase in corrosion levels [CITE Ma et al. 2012; Yang et al. 2016]. It's worth mentioning that the research listed above have only looked at the effects of corrosion on column flexural performance. The tested RC columns were either ductile or only longitudinal reinforcements were corroded [CITE Meda et al. 2014; Goksu and Ilki 2016], hence the mode of failure was flexure. As, the transverse reinforcement has a higher probability of being corroded before the longitudinal reinforcement due to the smaller distance to the corrosive atmosphere, there may be a significant change in the behavior of the columns having corrosion in both reinforcements. Experimental tests by Vu and Li (2018) [CITE] reveal that 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 under low-axial level. For very high axial loads, the change in failure modes occur at a very low corrosion level of transverse reinforcements. Li et al. (2018) [CITE] also reported that with an increase in corrosion level of transverse reinforcement pure shear failure was observed with considerable loss of strength and stiffness along with a more severe pinching effect. Thus, for lifetime seismic vulnerability assessment of older RC buildings, it is imperative that analytical model should capture the non-ductile shear failure mechanisms of RC columns.

Addressing the existing drawbacks and augmenting past research, this study aims to quantify the influence of corrosion deterioration on the lifetime seismic vulnerability of RC frames using robust analytical models that can capture critical failure modes of low-ductility RC frames. The present study is performed on a three-story three-bay RC moment resisting frame for which laboratory test data on structural performance under dynamic and cyclic loading for the as-built frame exists in literature. Based on the considered exposure condition, a probabilistic model of time-dependent corrosion deterioration of the case-study frame is developed followed by development of state-of-the-art finite element (FE) model that can capture critical failure modes typically observed in low-ductility RC frames. The FE model also incorporates the time-dependent corrosion deterioration effects. For confident seismic response predictions, FE model is validated against available experimental results of the frame and reversed cyclic tests of corroded RC columns having varied failure mechanisms (flexure, flexure-shear, and shear). The subsequent section presents non-linear time-history analyses (NLTHAs) results considering a large set of unscaled natural ground motion records to incorporate record-to-record variability within the seismic response. Comparison of seismic demand and capacity estimates obtained using pushover analyses for the as-built as well as corroded frame allows the development of seismic fragility curves for different damage states at different point in time along the service life of the building. Lastly, a generic fragility function is also presented to estimate probability of failure based on percentage mass loss measurements and ground motion intensity measure. The paper ends with key conclusions and recommendations for future explorations.

# FRAMEWORK FOR LIFETIME SEISMIC VULNERIBILITY ASSESSMENT

# CASE STUDY STRUCTURE: DESCRIPTION, DETERIORATION MECHANISMS AND FINITE ELEMENT MODELLING

The impact of corrosion deterioration on the lifetime seismic vulnerability of RC frame is demonstrated through a representative case-study RC frame assumed to be located in the coastal region of California, US. The case-study frame is non-seismically designed (low-ductility) and representative of typical constructions practices in several areas of the USA as well similar earthquake-prone regions in Europe, and Asia prior to the introduction of modern seismic design codes. Past experimental studies and earthquake investigation reports highlighted varied failure modes in these non-ductile frames [22,29]. Due to lack of seismic design, typically the failure modes in these buildings are related to strong beam/weak column hierarchy, with beams designed to resist the bending actions and columns sized to predominantly carry axial loads, and lack of seismic detailing, such as insufficient transverse reinforcement in columns and beam-column joints, inadequate anchorage length and hooks for reinforcement, lap splices in potential plastic-hinge regions of columns, among others. In addition to seismic hazard threats when located in close proximity to marine sources, the lateral load carrying capacity of these non-ductile buildings are further reduced during design service life due to corrosion deterioration. Corrosion of RC frame can cause significant reduction in the cross-sectional area of reinforcing steel and several secondary effects such as loss of cover and core concrete strength, reduction of yield or ultimate strength and ductility of steel reinforcement, and loss of bond strength, among others. Such effects significantly reduces the load-carrying capacity of the RC frame against the lateral load that may lead to a precarious brittle failure mechanism [11,33,34]. The following subsections first describe the representative case-study frame geometry. Next, the time-dependent deterioration modeling of the RC frame due to corrosion is discussed, followed by the finite element modeling approach.

## Description of the Case-Study Frame

Figure 1 shows the layout of the case study frame adopted from Bracci et al. [36]. The selected three-storey three bays RC moment resisting frame is representative of non-seismically designed (low-ductility) low-rise RC buildings. The frame has a total height of 10.75 m (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 building frame is designed for gravity loads only following the pre-seismic design provisions of ACI 318-89 [37]. The column sizes are 300 x 300 mm and the beam sizes are 230 x 460 mm at each floor. The concrete design compressive strength is 24 MPa, and the longitudinal reinforcing bar of steel Grade 40 has yield strength of 276 MPa. Further reinforcement details of the case study frame as shown in Figure 1 are adopted from Bracci et al. [36] and Aycardi et al. [38]. As evident from the figure, columns and beam-column joints have insufficient transverse reinforcement, inadequate anchorage length and hooks for reinforcement, and lap splices in potential plastic-hinge regions thereby highlighting lack of seismic detailing. The next subsection introduces the corrosion deterioration mechanism of the case-study frame.

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| Figure 1. Case-study frame layout adapted from Bracci et al. [CITE] |

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

When located in adverse environmental conditions, RC buildings are subjected to time-dependent aging and degradation. The aging and deterioration process of RC structures manifest in the form of corrosion, and other physical and chemical deterioration. Among these, corrosion deterioration due to the ingress of chloride ions constitutes a dominant form of degradation of RC structures [CITE Stewart and Rosowsky 1998a; Choe et al. 2009; Akiyama et al. 2011]. Sources of chlorides may stem from marine environments, or airborne chloride that constitute common exposure scenarios for RC buildings located in the coastal area of case-study region of California. Accordingly, this study considers chloride-induced corrosion as the primary form of time-dependent degradation of the case-study building frame. Corrosion deterioration in RC structures begins after a time interval known as the corrosion initiation time (*Tinit*). Chloride ions gradually penetrate the concrete cover during this time, depassivating the reinforcing steel and initiating corrosion. This study uses the widely adopted probabilistic model proposed by Duracrete [CITE] to predict corrosion initiation time represented as:



where, *cv* is the concrete cover depth in mm, *Dcl,*0 is the diffusion coefficient at *t*0 = 28 days in mm2/year determined from compliance test*, kt* is the correction factor of tests performed to estimate *Dcl*,0, *ke* and *kc* are environmental and curing factor, *ncl* is the age exponent considering densification of cement paste due to hydration chloride, *Cs* is the equilibrium chloride concentration at the exposed concrete surface, *Ccr* is the critical chloride concentration, and *erf* is the Gaussian error function. This study assumes marine splash environmental condition and probabilistic distribution of the parameters to estimate corrosion initiation time are adopted from Duracrete [CITE] and reported in Table 1.

**Table 1. Probability distribution of the parameters involved to estimate corrosion initiation time and rate of corrosion.**

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| ***Dcl*,0: Reference chloride diffusion coefficient for *w/c* ratio = 0.50** [39] | | | | | | |
| Distribution: *Normal* (𝜇, 𝜎) | 𝜇 | | | 𝜎 | | |
| 473.0 *mm*2/*year* | | | 43.2 × 10-12 *m*2/*s* | | |
| ***ke*: Environmental correction factor** [39] | | | | | | |
| Distribution: *Gamma* (𝛼, 𝛽)† | 𝛼 | | | 𝛽 | | |
| 2.92 | | | 11.0 | | |
| † *Gamma* (𝛼,𝛽) refers to Gamma distribution with shape parameter 𝛼 and inverse scale parameter 𝛽 | | | | | | |
| ***kc*: Correction factor for curing time** **(*At age* 7 *days*)** [39] | | | | | | |
| Distribution: *Beta* (*a, b, p, q*)‡ | *a* | *b* | *p* | | | *q* |
| 1.0 | 4.0 | 2.15 | | | 10.7 |
| ‡*Beta* (*a, b, p, q*) refers to a four-parameter Beta distribution with *a* and *b* representing the upper and lower bounds, and *p* and *q* representing the shape parameters | | | | | | |
| ***ncl*: Aging factor or age exponent** [39] | | | | | | |
| Distribution: *Beta* (*a, b, p, q*) | *a* | *b* | *p* | | | *q* |
| 0.0 | 1.0 | 17.2 | | | 29.3 |
| ***Ccr*: Critical chloride concentration (% relative to binder)** [39] | | | | | | |
| Distribution: *Normal* (𝜇, 𝜎) | 𝜇 | | | 𝜎 | | |
| 0.50 | | | 0.10 | | |
| ***Cs*: Surface chloride concentration (% relative to binder)** (Duracrete, 2000) | | | | | | |
| Calculated as *Cs* = *Acs* (*w/b*) + *εcs*, where *w/b* is the water-binder ratio (assumed as 0.5 in this study), *Acs* is chloride surface content regression parameter and *εcs* is the error term for surface chloride concentration. These two parameters are described below. | | | | | | |
| ***Acs*: Chloride surface content regression parameter (% relative to binder)** (Duracrete, 2000) | | | | | | |
| Distribution: *Normal* (𝜇, 𝜎) | 𝜇 | | | | 𝜎 | |
| 7.76 | | | | 1.36 | |
| ***𝜺cs* : Chloride surface content error term** (Duracrete, 2000) | | | | | | |
| Distribution: *Normal* (𝜇, 𝜎) | 𝜇 | | | | 𝜎 | |
| 0 | | | | 1.11 | |
| ***cv*: Concrete cover depth (mm)** | | | | | | |
| Distribution: *Lognormal* (*λ, ζ*) | ***λ*** | | | | *ζ* | |
| 3.62 and 3.84 | | | | 0.2 | |
| Parameters *λ* and *ζ* are the mean and standard deviation of the corresponding Normal distribution. The reported values of the parameters correspond to a mean cover depth of 38.1 and 47.61 mm for transverse and longitudinal steel respectively, and a coefficient of variation of 0.20 | | | | | | |
| ***icorr* : Corrosion rate (µA/cm2)** (Duracrete, 2000) | | | | | | |
| Distribution: *Lognormal* (*λ, ζ*) | ***λ*** | | | | *ζ* | |
| 0.766 | | | | 0.60 | |
| Parameters *λ* and *ζ* are the mean and standard deviation of the corresponding Normal distribution. The reported values of the parameters correspond to a mean corrosion current density of 2.586 **(**µA/cm2**)** and a coefficient of variation of 0.60 | | | | | | |

Considering uncertainty of the above-mentioned parameters, a Monte Carlo simulation is carried out to estimate the mean corrosion initiation time for longitudinal and transverse reinforcement of RC members. Figure 2 (a) shows the simulated corrosion initiation time for transverse steel reinforcement having mean cover depth of 38.1 mm. Using Lilliefors test, a lognormal distribution with mean of 10 years is found to be a good fit to the simulated data for corrosion initiation time of transverse steel reinforcement. In similar way, the mean corrosion initiation time for longitudinal steel reinforcement is estimated as 14 years. Note that a lower initiation time is expected for transverse steel as it is located closer to the outside environment as compared to longitudinal reinforcement (See column details in Figure 1).

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| **(a)** | **(b)** |
| **Figure 2 (a) Distribution of corrosion initiation time for transverse reinforcement, and (b) Variation of mean longitudinal and transverse steel area with time along with the lower and upper limits of the uncertainty band representing 5th and 95th percentile confidence bounds.** | |

Following the initiation phase, the corrosion propagation phase is marked by the formation of small independent pits or cracks along the steel rebar that with time lead to wider cracks resulting in uniform corrosion [40] [CITE]. The cross-sectional area of reinforcing steel at time *t* [*Ast*(*t*)] can be estimated as:



where, *D*0 is the diameter of the uncorroded reinforcing steel, *t* is the elapsed time and *Tinit* is the corrosion initiation time [Equation ] in years, and *icorr* is the corrosion current density in µA/cm2. While several corrosion rate models are available in the literature [CITE], to maintain consistency with corrosion initiation model, the corrosion rate is also adopted from Duracrete [CITE] and reported in Table 1. Note that non-uniform pitting corrosion of steel reinforcement is not studied in this work for the sake of simplicity. However, based on recommendations of Dizaj et al. [CITE] when modelling uniform corrosion other implications of non-uniform pitting corrosion such as the reduction in steel strength and ductility are explicitly included in this work as elaborated later. Figure 2 (b) shows the variation of mean reinforcing steel area (longitudinal and transverse) with time along with the lower and upper limits of the uncertainty band representing 5th and 95th percentile confidence bounds after incorporating the uncertainties in the corrosion deterioration process, as mentioned in Table 1. The effect of area loss of reinforcement is more pronounced for smaller diameters [CITE], thus higher percentage area loss is observed for transverse steel as evident in the figure. At the end of 50 years, 19% and 38% reduction in area loss is observed for longitudinal and transverse reinforcement of columns, signifying the necessity of considering corrosion deterioration in the lifetime seismic vulnerability assessment of RC buildings.

In addition to the primary effect of corrosion deterioration through area loss of longitudinal and transverse reinforcement, the present study also considers several secondary effects such as the reduction in concrete core and cover strength, and reduction in steel strength and ductility. Expansive rust products formed due to rebar corrosion results in the generation of micro-cracks that can lead to a reduction in cover concrete strength and even spalling of concrete over time. The deterioration of cover concrete strength is evaluated using the model proposed by [41][CITE]. The concrete core strength is a function of the volumetric ratio of transverse reinforcement (stirrups) and undergoes a time-dependent reduction due to corrosion deterioration. This time-dependent loss in core concrete strength is estimated based on the modified theoretical stress-strain model proposed by [42][CITE Mander 1988], as subsequently adopted by other researchers [CITE]. Reduction in steel mechanical properties due to corrosion deterioration along the rebar length are manifested through changes in yield strength, ultimate strength, and ultimate strain. This study utilizes Du et al. [CITE] time-dependent model for steel strength reduction, in which yield and ultimate strength decreases linearly with mass loss. In contrast, the ultimate strain reduction in steel does not follow a linear reduction, as past research revealed that strain reduction values are scattered over a large range [CITE Andisheh et al. 2016]. As a result, this research incorporates Apostolopoulos and Papadakis [CITE] experimental findings, which imply a nonlinear reduction in ultimate strain with percentage mass loss of reinforcing steel. At the end of service life (50 years), the deteriorated RC columns also undergo XX%, XX%, XX%, and XX% mean reduction in the cover concrete strength steel strength, ultimate strain, and shear strength, respectively (Figure 3), signifying the need to consider these secondary effects in the lifetime performance assessment of RC frame buildings.

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| **Figure 3**. **Time-dependent variation of columns cover concrete strength steel strength, ultimate strain, and shear strength** |

In addition to the above secondary effects, corrosion deterioration in RC columns may also result in bond strength reduction, buckling strength reduction, and low-cycle fatigue degradation, among others [CITE Fang et al. 2004; Kashani et al. 2015]. This study neglects the inclusion of loss of bond strength for deteriorating columns, since such effects have insignificant influence on the seismic fragility of RC frames [CITE]. The phenomenological model for capturing buckling strength and low-cycle fatigue degradation due to corrosion [CITE Kashani et al. 2015] are applicable only for corroded bare reinforcing steel, and further experimental tests are needed to verify this model for corroded RC members, consequently not considered in this study. The next section of the paper explains the analytical modeling of the case-study aging building frame.

## Analytical Modeling of the Case-Study RC Frame

A two-dimensional high-fidelity finite element model of the case-study frame is developed using the finite element (FE) package OpenSees [CITE McKenna, Fenves, & Scott, 2000]. The FE model strategy for the case-study frame follows the modelling suggestions presented in Freddi *et al.* [CITE 2021] that focus on better prediction of the local seismic response of structural components, including brittle failure mechanisms that are typical of low-ductility RC MRFs. The adopted high-fidelity modeling strategy enables modelling of local failure mechanisms that helps to monitor the influence of corrosion deterioration of building components on the global response parameters. The following sections first provide details of finite element modeling of the pristine frame followed by discussion on incorporation of deterioration effects in the FE modeling of corroded frame. The next sub-section provides validation of FE model with past experimental results.

### FE modeling of case-study frame incorporating corrosion deterioration effects

Figure 4 shows the schematic representation of the FE model highlighting modeling details of the different components such as beam and column sections, interior and exterior beam-column joints, and shear and axial springs. The non-linear flexural response of columns is captured using *nonLinearBeamColumn* element that considers the spread of plasticity along the length of the element. The column section is modeled using a fiber section that includes cover and core concrete patches and layers of reinforcement (Figure 4). Both unconfined cover concrete and confined core concrete are modeled using *Concrete02* material, and confined concrete parameters are calculated based on Mander et al. (1988) [CITE]. Steel reinforcements are modelled using the *uniaxialMaterial Hysteretic* material capable of capturing pinching of force and deformation, damage due to ductility and energy, and degraded unloading stiffness based on ductility. The pinching, strength, and stiffness degradation parameters for case-study frame columns are calibrated based on experimental result of Aycardi et al. [CITE] and mentioned in Freddi et al. [CITE 2013]. Beams are modeled using *beamWithHinges* elementthat consists of a central elastic element and two plastic hinge regions at the elements ends defined by fiber sections. The plastic hinge of beams is evaluated based on model proposed by Panagiotakos and Fardis [CITE], and fiber sections are defined similar to the approach adopted in columns with *Concrete02* material used to model core and cover concrete and *uniaxialMaterial Hysteretic* material used to model steel reinforcement. The slab is modeled using unconfined concrete material model with an effective width equal to four times the beam's width, as recommended in the ACI 318-89 [CITE]. The rigid-floor diaphragm is modeled by assigning high axial stiffness to the beams. Gravity loads are distributed on the beams while masses are concentrated at the beam-column intersections.

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| Figure 4. Overview of the numerical modeling strategy for case-study frame. |

For the deteriorated RC beams and columns, the consequences of chloride-induced corrosion, such as area loss of steel and various secondary effects outlined previously in Section 3.2, are incorporated along with the associated uncertainties. The effect of corrosion is assumed to be uniformly distributed around the perimeter of beams and columns. The area loss of steel due to corrosion is modeled as a uniform reduction in rebar diameter along the circumference and includes the corrosion rate time-dependent uncertainty (Table 1 and Figure 2). The finite element modeling of the degraded frame also includes the outlined secondary effects of corrosion deterioration, such as rebar strength and ductility loss, as well as loss of cover and core concrete strength. The loss of strength and ductility of rebars is captured by calculated the time-dependent strength and ductility of reinforcement [Figure 3], and updating the parameters of the *uniaxialMaterial Hysteretic* material used to model reinforcing steel. The time-dependent core concrete strength due to corrosion of transverse reinforcement is incorporated by updating the parameters of *Concrete02* material model used to define the column core. The loss of cover strength due to the formation of expansive rust products is modeled by calculating the time-dependent strength of cover concrete [Figure 3], and then updating the parameters of *Concrete02* material used to define column cover concrete.

While the above fiber section-based modeling of a RC section is capable of capturing the nonlinear flexural deformation, non-seismically designed low-ductility frames may also exhibit nonlinear behaviour due to column shear failure and loss of gravity load-bearing capability [CITE Elwood 2004]. Consequently, a zero-length shear spring (element *zeroLength* in OpenSees) is placed in series with the column flexure element as shown in Figure 4. The *uniaxial* *LimitState* material model is assigned to these springs that monitors columns response and triggers only when column response reaches the pre-defined shear and axial failure curves, implemented within OpenSees as *limitCurve Shear* and *limitCurve Axial*, respectively. Note that the existing *limitCurve Shear* model is modified such that shear failure is triggered when column response reaches either the strength or drift limit curves [CITE Freddi et al. 2021]. While the shear strength limit curve is based on the model proposed in ASCE-41, the deformation limit curve is defined based on an empirically derived force-deformation controlled limit curve given by Elwood [CITE]. Both strength and drift shear limit curves depends on parameters such as column transverse reinforcement, column axial load, section dimensions, material properties, among others, and therefore, are affected by column corrosion deterioration. The considered FE modeling approach considers this degradation of shear limit curves during the nonlinear analysis. The degrading slope of the total response (*Ktdeg* in Figure 4) is calculated using the shear-friction model proposed by Baradaran-Shoraka and Elwood [CITE]. The degrading stiffness of shear spring response (*Kdeg* in Figure 4) is estimated from the degrading slope of the total response *Ktdeg* and the unloading stiffness of flexural response (*Kunload*) [CITE Elwood 2004]. Also, the residual strength is defined as 20 percent of the nominal shear strength to alleviate convergence issues in OpenSees [CITE Jeon et al. 2015].

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. [CITE], and shown in Figure 4. 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 suggested by Jeon et al. [CITE]. Also, to account for the pinching behavior *Pinching4* material model [CITE] is used in the beam-column joint response as shown in Figure 4. The present study doesn’t consider influence of corrosion deterioration of joints on seismic response and is left for future exploration.

### Validation with past experimental studies

For the purpose of validating the OpenSees [24] 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)*. 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 [19]. 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 up to 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 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.

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| **(a)** | **(b)** |
|  |  |
| **(c)** | **(d)** |
| **Figure 5 (a)** | |

The subsequent section will use the validated analytical model with parameters calibrated from the experimental results for both joints and columns to develop capacity and demand models for seismic fragility analysis.

# TIME-DEPENDENT SEISMIC FRAGILITY ANALYSIS

Seismic fragility curves are developed for four different damage states. At any point in time along the service life of the building, seismic fragility curves are developed after comparing the seismic demand and capacities of nominally similar but statistically different finite element frame models. These models reflect the deterioration uncertainty pertaining to corrosion deterioration of the frame. Seismic fragilities are developed at 0 years (pristine), 25 years, and 50 years. The limit state capacities of frame are derived via non-linear static pushover analysis of bridge columns considering the aleatoric and epistemic uncertainty associated with modeling of the frame using a Monte Carlo approach.

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| **Figure 6**. |

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| **Figure 6**. |

# SEISMIC FRAGILITY SURFACE FOR GENERIC

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| **Figure 6**. |

# APPENDIX

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

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Specimen | U2 | C5 | CC2 | S1 |
| Reference | Vu and Li (2018a) [19] | Vu and Li (2018a) [19] | Vu and Li (2018b) [18] | Sezen and Moehle (2006) [43] |
| 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 |

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